Basis of Design Report– Remedial Design
Elm Avenue Storm Drain Relocation and Groundwater Collection Trench
Atlantic Wood Industries (AWI) Superfund Site
Portsmouth, Virginia

Remedial Action Contract 2
Contract: EP-S3-07-07
Work Assignment: 011RDRD03L2

Prepared for

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August 2012
Revision: 01
EA Project No. 14530.11

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LIST OF ACRONYMS AND ABBREVIATIONS

ARAR	Applicable or Relevant and Appropriate Requirements
AWI	Atlantic Wood Industries
BTEX	Benzene, Toluene, Ethylbenzene, and Xylenes
cfd	Cubic Feet Per Day
cfs	Cubic Feet Per Second
COC	Contaminant of Concern
DNAPL	Dense Non-Aqueous Phase Liquid
EA	EA Engineering, Science, and Technology, Inc.
EPA	U.S. Environmental Protection Agency
FIGG	FIGG Bridge Developers, LLC
ft	Foot or Feet
gpm	Gallons Per Minute
HDPE	High Density Polyethylene
HEC-RAS	Hydrologic Engineering Center Riverine Analysis System
in.	Inch(es)
LF	Linear Feet
NAVD88	North American Vertical Datum of 1988
NPDES	National Pollutant Discharge Elimination System
OU	Operable Unit
OSPW	Offshore Sheet Pile Wall
PAH	Polycyclic Aromatic Hydrocarbon
PCP	Pentachlorophenol
POI	Point of Investigation
RA	Remedial Action
RAGS	Risk Assessment Guidance for Superfund
RAWP	Remedial Action Work Plan
RCN	Runoff Curve Number
RD	Remedial Design
ROD	Record of Decision
RSL	Regional Screening Level
SCS	Soil Conservation Service
TC	Time of Concentration

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LIST OF ACRONYMS AND ABBREVIATIONS (continued)

USACE	United States Army Corps of Engineers
VDEQ	Virginia Department of Environmental Quality
VDOT	Virginia Department of Transportation
VOC	Volatile Organic Compound
VPDES	Virginia Pollutant Discharge Elimination System
VPA	Virginia Pollutant Abatement

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1.0 INTRODUCTION

Under U.S. Environmental Protection Agency (EPA) Remedial Action Contract No. EP-S3-07-07, EA Engineering, Science, and Technology, Inc. (EA) has prepared this Remedial Design (RD) Report for Work Assignment 0011RDRD03L2 for the remedial design of the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench at the Atlantic Wood Industries (AWI) Superfund site in Portsmouth, Virginia.

This RD report documents the basis of design performed for the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench and how the RD meets the requirements set forth in the Record of Decision (ROD) (U.S. EPA 2007). This design includes the relocation of the Elm Avenue storm drain, future cap drainage, and groundwater collection trench as part of the RD.

The storm drain design includes the relocation and extension of the existing storm drain system on the south side of Elm Avenue, the outfall of which will be obstructed by the construction of the Offshore Sheet Pile Wall (OSPW). The design also includes the evaluation for future installation of stormwater quality facilities to manage the increase in impervious area from the ultimate development of the dredged material containment facility. The stormwater quality facilities will be constructed as part of a future remedial action (RA) effort.

For this phase of design, EA is also including a groundwater collection trench to provide hydraulic control of groundwater on both the east side of the AWI property (east of Burton's Point Road) and the inboard side of the OSPW following its construction. EA is proposing to include the groundwater collection component with the design and installation of the storm drain relocation to create cost and time savings for EPA. This inclusion provides an increase in construction efficiency by combining two systems required in the same vicinity into one construction contract. The groundwater collection trench proposed under this phase of the design represents only the initial portion of the overall groundwater management system for the site. The remainder of the groundwater management system will be developed as part of a future RD effort and will provide additional hydraulic control and potential treatment of groundwater.

This report describes the design intent, project background, analyses and details associated with the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench design.

1.1 PROJECT DESCRIPTION

The AWI site consists of approximately 48 acres of land on the industrialized waterfront area of Portsmouth, Virginia. From 1926 to 1992, a wood treating facility operated at the site using both creosote and pentachlorophenol (PCP). The facility operations included wood treatment, storage of wood, and disposal of wastes, which lead to the contamination of the site. At one time, the Navy leased a portion of the property from AWI and disposed of waste onsite, including used abrasive blast media and calcium hydroxide sludge. As a result of historical site operations, sediments in the Elizabeth River contain visible creosote. The groundwater and soil at the site are also heavily contaminated with creosote. Creosote contamination previously migrated into a storm sewer and discharged to an inlet of the Elizabeth River at the northeast corner of the site near the former Jordan Bridge (Virginia Route 337).

Currently, AWI (now known as Atlantic Metrocast, Inc.) operates a pre-stressed concrete products manufacturing facility on the site. Groundwater in this area is not used as a drinking water source.

EPA selected a remedy for the site in the December 2007 ROD which established performance standards for each of the three operable units (OU1, OU2, and OU3) at the Site and specified remedies that addressed soil, sediment, and groundwater contamination.

Due primarily to funding considerations, EPA elected to separate the remedial design into a phased approach, roughly based on the ROD remedy components. Each phase will have a separate design package to be prepared by EA. Two phases have been proposed: Phase 1 designs were completed in 2009-2010; portions of the Phase 2 design were completed in 2010-2011. The remaining remedy components and design features to be completed for Phase 2 of the RD include the following:

- Elm Avenue Storm Drain Relocation
- West Side Containment Berm Completion
- Site Capping
- Stormwater Management/Drainage
- Erosion and Sediment Control
- Dredging and Dredged Material Handling
- Hydrogeologic Analysis
- Groundwater Management
- Operation and Maintenance Plans

Figure 1 illustrates the various remedy components for the AWI site. Please note that some of the components are constructed, some are currently under construction, and some are currently in design.

This report focuses on the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench. To date, the major design development and review milestones for this RD have included:

- Groundwater Management Alternatives Analysis Submittal – January 2011
- Preliminary Design documents placed on EPA Environmental Science Connector for stakeholder access and review – December 2011
- Pre-Final Design documents placed on EPA Environmental Science Connector for stakeholder access and review – March 2012

1.2 BACKGROUND

The project site is located in a low-lying area bordering the Southern Branch of the Elizabeth River. Elevations range from sea level along the river to approximately 9.5 feet (ft) above mean sea level. The majority of the site falls within the 100-year floodplain, with the exception of the western edge of the property. Burton's Point Road is slightly elevated above the surrounding area and acts as a divide for surface water drainage, separating the AWI property into eastern and western drainage areas. Surface water within the west side drainage area flows into a drainage ditch along the western property boundary; stormwater then enters a storm drain system and is ultimately discharged to Paradise Creek (a tributary to the Southern Branch of the Elizabeth River) located south of the site. Surface water east of Burton's Point Road generally flows toward the Elizabeth River with some flows intercepted and conveyed through the existing storm drain along Elm Avenue. Surface water from the properties north of Elm Avenue is intercepted by an existing 15-inch (in.) storm drain system located within Veneer Road with inlets along the east and west sides of Veneer Road conveying flows south to the existing 24-in. storm drain along Elm Avenue.

The overall remedy includes the construction of the OSPW and the recently completed East Side Containment Berm (construction completed November 2011), which constitute portions of a containment facility for the placement of contaminated sediment to be dredged from the Southern Branch of the Elizabeth River. The construction of the containment facility will obstruct the discharge of the existing Elm Avenue storm drain system and ultimately generate new land, which will produce increased stormwater runoff. To accommodate the existing stormwater flows and future flows generated by the newly created land, a preliminary analysis of the Elm Avenue Storm Drain extension was performed in conjunction with the design of the OSPW. The design resulted in the inclusion of three (3) storm drain penetrations in the OSPW that will serve as the discharge point for the proposed storm drain system extension.

As a result of the construction of the OSPW barrier, groundwater will accumulate behind and migrate around the sheet pile wall if uncontrolled. As part of the overall RD, it will be necessary to control, convey, and treat (if necessary) the groundwater accumulated behind the wall. The 2007 ROD requires hydraulic control of the expected mounding of groundwater behind the wall and includes specific control methods for evaluation. The initial step to identify the hydraulic control options was the assessment of the quantity of groundwater through modeling. The next step evaluated suggestions noted in the ROD, as well as other viable options, to hydraulically control the quantity of groundwater identified in the modeling. The goal of the ROD and EA's preliminary groundwater management design was to consider and incorporate hydraulic control and passive groundwater treatment (if practicable) to the greatest extent possible. The results of the analysis are described in the Draft Groundwater Management Alternatives Analysis[1]. This alternatives analysis presents feasible methods for controlling the groundwater at the AWI site.

[1] Atlantic Wood Industries (AWI) Superfund Site – Phase 2 Remedial Design, Groundwater Management Alternatives Analysis, EA Engineering, Science, and Technology, November 2011.

1.3 STAKEHOLDER COORDINATION

Section 11.2.5 of the ROD (U.S. EPA 2007) requires coordination with the AWI property owner and adjacent property owners to minimize disruptions to ongoing business operations. Specific actions regarding stakeholder involvement include:

- Minimize the disruptions to AWI's ongoing pre-cast concrete manufacturing operations;
- Coordinate with the property owners of the 3975 Elm Avenue property and the PER property to minimize disruption of redevelopment activities on their respective properties;
- Coordinate with FIGG Bridge Developers, LLC (FIGG) regarding activities around the former Jordan Bridge (previously owned by the City of Chesapeake) and the construction of the South Norfolk Jordan Bridge; and
- Coordinate with the City of Portsmouth Public Works Department, which will provide routine operation and maintenance on the relocated Elm Avenue storm drain.

Substantial coordination efforts have been performed by EPA and EA throughout the design process. Stakeholder input was actively sought by EPA/EA and design considerations were made in response to that input to minimize disruptions to ongoing and future business operations. Coordination was performed with the following stakeholders during design:

- AWI
- The 3975 Elm Avenue Property Owner
- The PER Property Owner
- FIGG
- The City of Portsmouth
- The Virginia Department of Environmental Quality (VDEQ)

EPA/EA performed extensive coordination with many stakeholders through meetings, telephone calls, and e-mails; special efforts and additional emphasis were placed on the stakeholders upon whose property the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench project will be constructed. The more significant coordination efforts with the stakeholders are summarized in the following table.

Elm Avenue Storm Drain Relocation and Groundwater Collection Trench Significant Coordination Events		
Date	**Coordination Description**	**Stakeholders Included**
August 9, 2011	EA met in Portsmouth, Virginia with the Department of Public Works to discuss the project concept and requirements and comments proposed by the City.	• City of Portsmouth
December 14, 2011	The Preliminary Design was sent to stakeholders for their review and comment.	• AWI • 3975 Elm Avenue Property Owner • City of Portsmouth • PER Property Owner • VDEQ
January 13, 2012	EA met in Portsmouth, Virginia to discuss the Preliminary Design and the concerns relative to the property.	• 3975 Elm Avenue Property Owner
February 16, 2012	EA requested gantry crane specifications related to ongoing work operations conducted on AWI property.	• AWI
February 24, 2012	EA requested pier locations and geotechnical information related to the South Norfolk Jordan Bridge in an effort to avoid the piers, provide future maintenance access, and utilize existing soil information.	• FIGG
March 13, 2012	EA met in Virginia Beach, Virginia to discuss ongoing development design of PER Property.	• PER Property Engineer
March 30, 2012	The Pre-Final Design was submitted to stakeholders for review and comment.	• AWI • 3975 Elm Avenue Property Owner • City of Portsmouth • PER Property Owner • FIGG • VDEQ
May 9, 2012	EA met in Portsmouth, Virginia to discuss the Pre-Final Design and existing site conditions on the AWI Property.	• AWI

Elm Avenue Storm Drain Relocation and Groundwater Collection Trench Significant Coordination Events		
Date	**Coordination Description**	**Stakeholders Included**
June 27, 2012	The Final Design was submitted to stakeholders for review and comment. No comments were received.	• AWI • 3975 Elm Avenue Property Owner • City of Portsmouth • PER Property Owner • FIGG • VDEQ

Note: The above table is not meant to be an all-inclusive list of coordination activities.

General descriptions of issues that may impact specific stakeholders and related design modifications are described in the following sections.

1.3.1 AWI

EPA coordinated with AWI during preparation of the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench RD. EPA provided the contract documents to AWI to solicit comments. The coordination with AWI resulted in:

- Change in alignment of the proposed storm drain and groundwater trench to allow access to the southern bulkhead.
- Increased structural design of the pipe and stormwater structures based on AWI equipment requirements.
- Coordination of construction sequence to lessen impact to ongoing business activities.
- Requirement of a pre-construction topographic survey of the project area to account for placed fill associated with the South Norfolk Jordan Bridge.

1.3.2 3975 Elm Avenue Property Owner

EPA coordinated with the owner of the 3975 Elm Avenue property during preparation of the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench RD.

EPA provided the design documents to the owner of the 3975 Elm Avenue property to solicit comments. The significant design modifications that resulted from coordination with the 3975 Elm Avenue property owner were:

- In the future, if a stormwater quality structure is necessary, it will be placed in an area that will minimize or eliminate the encroachment onto the 3975 Elm Avenue property.

EPA also considered access and future beneficial use of consolidated dredged material containment area upon completion of the dredged material placement for the owner of the 3975 Elm Avenue property. Accordingly, EPA/EA will provide a drivable transition from the current grade on the 3975 Elm Avenue property to the top of the new land at elevation 10.5 (feet, North American Vertical Datum of 1988 [NAVD] 88). **The drivable transition will be designed as part of a future RD effort.**

1.3.3 PER Property

EPA coordinated with PER during preparation of the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench RD.

EA provided the design documents to PER to solicit comments. The coordination with the PER property owner resulted in:

- Revised grading design of the future dredged material on PER property to accommodate PER development plans.
- Removal of the stormwater quality structure on PER property proposed to manage stormwater quality requirements for the property. PER development plans will include stormwater quality management for the newly generated land.

1.3.4 FIGG Bridge Developers, LLC

EPA coordinated with FIGG during preparation of the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench RD. EPA provided the design documents to FIGG to solicit comments. The coordination with FIGG resulted in:

- Change in alignment of the proposed storm drain to avoid conflict with future bridge piers.
- Obtaining supplemental geotechnical information on existing site soils.

2.0 BASIS OF DESIGN

The basis of design report provides a description of the analyses conducted in the development of the design approach. The following sections provide discussion of the design assumptions, the RA contracting strategy, regulatory requirements, and the identification of easement and access requirements.

In reviewing this report, the following factors must be considered as major objectives and/or constraints of this design;

1. The relocated Elm Avenue storm drain system is designed to convey 10-year, 24-hour design storm flow from the existing drainage areas up stream of the Elm Avenue − Veneer Road intersection and the anticipated runoff from the new land that will be created by the placement of contaminated Elizabeth River sediments behind the OSPW. Due to the flat site topography, the amount of impervious area in the local drainage areas, and the water level of the Southern Branch of the Elizabeth River, it is not possible to accommodate drainage of runoff during a storm surge condition.

2. The relocated Elm Avenue storm drain system must discharge to the pre-placed 42-in. outfalls which penetrate the Offshore Sheet Pile Wall near the restored wetlands. The construction of these pre-placed outfalls is part of the Offshore Sheet Pile Wall remedial action and is not part of the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench project. The pre-placed outfalls are currently in construction under the OSPW Contract.

3. The City of Portsmouth, who will be responsible for the maintenance of the relocated Elm Avenue storm drain system, has requested that the design minimize or eliminate exposure of city maintenance workers to contaminated environments and materials to the greatest extent possible. Therefore, the relocated Elm Avenue storm drain system includes a geomembrane liner to provide protection to their employees and minimize exposure to contaminated material.

4. Between Junction boxes 1 and 2, the relocated Elm Avenue storm drain system must be able to withstand the loads generated by AWI's fully loaded gantry cranes. It is not anticipated that these gantry cranes will be in operation north of Junction Box 2.

Although many additional, more detailed design criteria are developed and presented in this document, the reader should keep in mind these four main considerations when reviewing this Basis of Design report.

It is the intent of this document to be reviewed with the contract drawings as frequent references to the drawings are included in this report. Having both documents available will give the reader a better understanding of the site and the project.

2.1 ELM AVENUE STORM DRAIN SUMMARY

The existing Elm Avenue storm drain system currently discharges into a small inlet along the south side of Elm Avenue which outfalls into the Southern Branch of the Elizabeth River. Portions of the existing

Elm Avenue Storm Drain have been fiberglass in-situ form-lined to minimize the infiltration of contamination into the pipe and eventually into the river. Working around the in situ form liner installed in the storm drains around the intersection of Elm Avenue and Veneer Road will require care during demolition and connection to the proposed storm drain.

The obstruction created by the installation of the OSPW makes it necessary to relocate this system along the western edge of the future dredged material containment cell and extend it to a new point of discharge through the OSPW. The existing discharge point and proposed storm drain alignment are illustrated in *Figure 3 – Proposed Conditions Plan*. The need for this new discharge point led to the decision to include steel pipes through the southern bulkhead portion of the OSPW as part of the OSPW Contract Documents, which were issued for construction by the USACE Norfolk District.

The design of the southern bulkhead includes the use of a concrete deadman tieback system which is designed with a bottom elevation of 0.0 (NAVD88). The 48-in. king piles placed as part of the OSPW have a clearance of 4 ft − 10 in. between the king piles. Based on the design of the OSPW, the outside diameter of the steel pipes placed as part of the OSPW contract is 42 in. In order to avoid conflict with the concrete deadman, the steel pipes were designed at an invert elevation of -4 ft (NAVD 88). The steel pipes will be constructed to extend upstream beyond the limits of the concrete deadman which will allow the Elm Avenue storm drain relocation contractor to connect to the steel pipes and avoid the sheet pile wall components (pilings, deadmen, tiebacks, etc.). The size, location, and installation of the steel pipes are specified in the Contract Documents for the OSPW. The extent of the steel pipes and other components installed as part of the OSPW are shown in plan and profile view on Drawing C-5 STORM DRAIN PLAN AND PROFILE I of the Contract Drawings.

The portion of the existing storm drain along Elm Avenue and Veneer Road to be connected to the proposed storm drain relocation has a flat slope and shallow depth due to its proximity to the Southern Branch of the Elizabeth River and lack of relief across the watershed. The existing Elm Avenue storm drain, including size and invert information, is shown on Drawing C-2 EXISTING CONDITIONS PLAN of the Design Drawings. The shallow depth of the existing storm drain restricts the relocated pipe size that can be installed with adequate ground cover to a maximum diameter of 36 in. Due to this restriction, the design requirement to convey the 10-year, 24-hour design storm (per City of Portsmouth storm drain requirements) and the desire to minimize flooding during frequent storm events, multiple 36-in. pipes are required to convey stormwater runoff to the proposed discharge point.

Hydrologic analyses were performed for pre-development and post-development drainage conditions for the drainage areas to both the existing and proposed storm drain system. The assumptions and analysis summaries are presented in Section 2.2 of this report. The drainage area maps and hydrologic analyses are included in Appendix A. The analyses determined that three 36-in. pipes are required to convey the 10-year, 24-hour peak runoff for the downstream portion of the storm drain and two 36-in. pipes are required for the upstream portion. The existing and proposed storm drain systems, along with other design features, are shown on Drawing C-4 PROPOSED CONDITIONS PLAN of the Design Drawings.

<u>Storm Drain Alignment</u>

The alignment for the storm drain relocation and extension was chosen based on the need to avoid the existing bridge piers and to avoid potential future piers. The alignment was also chosen to be outside the limits of the future dredged material containment facility. The proposed alignment provides access points for connection to the future stormwater quality facilities required for runoff from the area created by the completed dredged material containment cell.

Operation and Maintenance

The existing storm drain is owned by the City of Portsmouth Public Works Department, which provides routine maintenance. Connecting to and extending the storm drain southeast across the AWI property will require a City of Portsmouth easement for future maintenance access. In order to avoid contact with contaminated soils during future maintenance activities, a geomembrane is included in the design to line the excavated pipe trench. To reduce potential damage to the geomembrane during maintenance of the pipe, separate vertical and horizontal easements are proposed. The horizontal easement will provide access for maintenance crews and equipment, while the vertical easement will restrict excavation access to the pipes only, thus restricting the potential for contact with the trench liner and underlying contaminated material. Cross sections of the proposed storm drain trench are illustrated on Drawings C-8 through C-10 of the Design Drawings.

To minimize the potential for flooding to occur at the site, the following considerations have been incorporated into the design:

Effects of Localized Storm Events, Storm Surges and Tidal Elevations

The project site is located within the coastal plain; therefore, the water surface in the adjacent Southern Branch of the Elizabeth River is tidally-influenced. Generally in coastal regions, large storm events (25-year, 50-year, and 100-year) are associated with hurricanes (Nor'easters) and tropical storms which generate large rainfall amounts resulting in large storm surges in the river, in addition to large stormwater runoff from upland areas. These storm surges in the river create significant flooding of the low-lying areas on the AWI and adjacent properties by inundation from the rising water surface of the Elizabeth River. More frequent storm events (2-year, 5-year, and 10-year) may not cause significant surges in river elevations. However, it is possible for the river to reach flood elevations as a result of weather occurring away from the site.

During these more frequent storm events; runoff will be captured and conveyed by the storm drain when the site is not experiencing a storm surge in the river, with only the tide creating a backwater condition. The water surface elevation of the river under storm surge conditions may inundate portions of the site regardless of the storm drain, i.e. the relocated Elm Avenue storm drain will not drain freely and will not prevent flooding during storm surge conditions. Because of this, the hydraulic models do not include river water surface elevations resulting from storm surge as a tailwater condition for the storm drain. Model scenarios were run using Mean High Water, Mean Sea Level, and Mean Low Water tide elevations as tailwater conditions. The description and results of the hydrologic and hydraulic analyses for the pre-development and post-development conditions are stated in Section 2.2 of this document.

Installation of Storm Drain Tide Valves

Because the proposed storm drain discharge point in the OSPW is located at elevation -4 to avoid the structural components of the OSPW, the outlet will be underwater and will be affected by fluctuations in the tide elevations of the Elizabeth River. Temporary, bolted-on blind flanges will be installed as part of the OSPW construction contract on the upstream end of the steel pipes penetrating the OSPW.

As part of the Elm Avenue Storm Drain Relocation construction, storm drain tide valves will be installed in the three steel pipes immediately downstream of the Storm Drain Junction Box 1. The storm drain tide valves will provide back flow prevention for the discharge point that will almost always be underwater. The design of the storm drain tide valves allow a head differential of approximately 1 in. across the valves to open the valves and allow the storm drain to discharge even in submerged conditions. The storm drain tide valves are shown in the storm drain profile located on Drawing C-5 STORM DRAIN PLAN AND PROFILE I and in the details on Drawing C-13 STORM DRAIN & GROUNDWATER COLLECTION TRENCH DETAILS of the Design Drawings.

2.1.1 Design

Details of the storm drain design are discussed in this section, including:

- Dewatering
- Grading/Excavation
- Material

2.1.1.1 Dewatering

The proximity of the Elm Avenue Storm Drain Relocation project to the Southern Branch of the Elizabeth River results in groundwater elevations near the existing ground surface. Due to these shallow groundwater elevations, the construction of the storm drain system will require dewatering to create workable conditions within the areas of excavation. A water control plan for the dewatering effort is required to be completed by the RA subcontractor in accordance with the specifications for this project. A groundwater model has been created to determine the quantity of water anticipated to be dewatered during excavation and construction activities. The description, illustrations, and results of the model have been included in Appendix B – Dewatering Calculations. Excavation activities will be sequenced to minimize the quantity of dewatering (and potential treatment) prior to local discharge. Requirements for dewatering including monitoring, potential treatment and discharge are included in the Contract Specifications. Available soil characteristics and water quality data in the vicinity of the proposed storm drain relocation is provided in Attachments 1 and 2 of the Contract Specifications.

2.1.1.2 Grading/Excavation

The ongoing AWI operations and current FIGG bridge construction project have changed the existing topography shown on the Design Drawings from the 2009 Woolpert survey. The proposed interim grading that has been designed as part of the Elm Avenue Storm Drain Relocation project has been graded to the existing topography as shown on the 2009 Woolpert survey. This interim grading will match the proposed grading around the southern bulkhead concrete deadmen that will be completed as part of the OSPW construction. The interim grading is shown on Drawing C-4 PROPOSED CONDITIONS PLAN of the Design Drawings. In order to minimize the amount of future grading, the interim grading specified over the storm drain alignment also conforms to the grades that are shown in *Figure 2 – Final Grading Plan*.

Contaminated Excavated Material

The AWI property was previously used for the treatment of wood products which has resulted in soil and groundwater contamination. All material excavated for the installation of the Elm Avenue Storm Drain Relocation will be considered contaminated and handled in accordance with Section 02 61 13 EXCAVATION AND HANDLING OF CONTAMINATED MATERIAL. Contaminated excavated material will be placed in the Stockpile Area A on the west side of the AWI site.

2.1.1.3 Material

The materials associated with the construction for the storm drain design include the following:

a. HDPE Geomembrane Liner

   The storm drain trench will be lined using a 60-mil textured HDPE geomembrane liner to minimize the potential for contaminated groundwater to enter the trench and backfill. The geomembrane liner also provides a barrier which will create an area within the trench that is considered clean and will allow the City of Portsmouth to maintain the storm drain without the need for specialized training. Calculations for the geomembrane liner have been prepared to illustrate the strength and anchor trench design and are included in Appendix C – Geomembrane Liner Calculations. Documentation from the Plastics Design Library regarding the high chemical resistance of the HDPE geomembrane liner to creosote is also provided in Appendix C.

b. Geotextile

   The HDPE Geomembrane liner will be lined along the top and bottom surfaces with two layers of 16-ounce, non-woven geotextile in accordance with AASHTO M 288. The bottom layer of geotextile will provide a cushion layer between the geomembrane and the underlying soil material. The top layer will provide a cushion layer between the geomembrane and the trench backfill for the storm drain; this additional geotextile layer will provide increased protection against puncture of the geomembrane. Geotextile will also be placed between the Select Bedding and Select Fill in the storm drain trench.

c. Storm Drain Junction Boxes

   The storm drain junction boxes have been designed to withstand HS-20 loading in accordance with Virginia Department of Transportation (VDOT) standards. The storm drain junction boxes will be protected with concrete bollards to restrict the gantry cranes and other equipment used by AWI from driving over the storm drain junction boxes. This restriction removes the need to design the structures for severe loading conditions and reduces the cost of the structures. The junction boxes are designed using lightweight concrete with a slab that will be below the geomembrane liner and will potentially be subjected to contact with contaminated soil. Design calculations for the storm drain junction boxes are included in Appendix D – Storm Drain Structural Calculations.

d. Reinforced Concrete Pipe

The storm drain pipes have been designed to withstand the loading conditions of the gantry cranes used for AWI operation and require installation of Class 3 reinforced concrete pipe. Design calculations for the storm drain are located in Appendix D – Storm Drain Structural Calculations.

e. Backfill and Grading Materials

The material needed for the backfill of the storm drain section and final grading will be clean, imported material meeting select material criteria required by the Hampton Roads Planning District Commission Regional Construction Standards.

The trench backfill will be saturated along the entire length of the relocated storm drain prior to placement of the low permeability gravel using clean, potable water.. By saturating the backfill, hydraulic pressure will be equalized between the inside and outside of the trench liner, minimizing the potential for contamination to migrate through the liner. Temporary observation wells will be installed prior to saturation to monitor the level of the saturation until saturation is achieved to the top of the select fill material. The observation wells will be abandoned in place once full saturation is achieved.

f. Storm Drain Trench Drains

Trench drains have been designed in the bottom corners of the trench upstream of Storm Drain Junction 1 for a distance of approximately 210 ft . The trench drains will consist of perforated 6-inch diameter HDPE pipe wrapped in a filter sock to minimize fines from entering the drains. The 6-inch HDPE pipe will transition to an 18-inch diameter HDPE pipe near Storm Drain Junction Box 1. The trench drains will include a monitoring port connected to the 18-inch diameter HDPE pipe and extended to the ground surface that will allow for the monitoring of water level and quality within the trench section. The trench drains will connect to Storm Drain Junction Box 1, but will remain capped to restrict discharge of the water within the trench into the junction box. Connection to the junction box will be for future evacuation of the water within the trench if necessary for future maintenance. .

Between Junction Boxes 1 and 2, there are two additional 6-inch diameter HDPE trench drains to be located just above the estimated elevation of the groundwater following ultimate site development. These two pipes are intended to keep the water level within the trench below the maximum desired groundwater level following completion of the AWI remedial action. The higher trench drains discharge to Junction Box 1 above the level of tidal influence.

Locations of the storm drain trench drains are illustrated on Drawing C-5 Storm Drain Plan and Profile I and details are located on Drawing C-13 Storm Drain & Groundwater Collection Trench Details.

2.2 ELM AVENUE STORM DRAIN DRAINAGE

2.2.1 Hydrologic Analysis

EA has evaluated both pre-development and post-development conditions for the drainage areas affected by the construction of the OSPW. The intent of this analysis is to identify the drainage areas contributing to the site, assess the existing stormwater infrastructure currently in place, and identify the requirements for stormwater management as set forth by the Applicable or Relevant and Appropriate Requirements (ARARs). In order to calculate peak stormwater discharges necessary to complete the Elm Avenue storm drain relocation design, EA utilized existing topographic survey to determine existing drainage areas and conditions (See *Figure 1 – Pre-Development Drainage Area Map and Figure 2 – Post-Development Drainage Area Map,* Appendix A).

The 2-year 24-hour, 5-year 24-hour, 10-year 24-hour, 25-year 24-hour, 50-year 24-hour, and 100-year 24-hour storm events were evaluated to determine the hydraulic capacity of the storm drain, in addition to the effects of the tidal water surface elevations on this capacity. Peak flows were calculated for the 2-year 24-hour, 5-year 24-hour, 10-year 24-hour, 25-year 24-hour, 50-year 24-hour, and 100-year 24-hour using the TR-55 (Technical Release – 55) hydrologic methodology built into Autodesk Storm and Sanitary Analysis Software. Technical Release – 55 is based on the Natural Resources Conservation Service (NRCS) formerly known as the Soil Conservation Service (SCS) method for computing peak flows. The method utilizes Runoff Curve Number (RCN)[2] and Time of Concentration (TC)[3] information as input for calculating peak flows for various design storms. Rainfall data included in the Autodesk Storm and Sanitary Analysis software is based on values from Technical Paper No. 40 (TP-40) released by United States Department of Commerce. Rainfall values from TP-40 for the region where the project site is located are higher than values from the newer National Oceanic and Atmospheric Administration (NOAA) Atlas 14 data and are therefore considered to be conservative. Rainfall distribution used for the hydrologic analysis is Type III distribution for the Virginia coastal region. The storm events were evaluated under three (3) tidal scenarios: Mean High Water, Mean Sea Level, and Mean Low Water. Land use boundaries and time of concentration paths were approximated using the existing topography as previously described.

[2]Runoff Curve Number, abbreviated RCN or CN, is calculated based on characteristics of the contributing drainage area, including the following parameters: Hydrologic Soil Group, Ground Cover, and Land Use.
[3] Time of Concentration, measured in units of hours, is calculated for each drainage area and based on the amount of time it would take for stormwater to travel on the longest flow path before outletting at the point of investigation (POI).

Table 1 – Drainage Area and Peak Flow Results

Sub Basin ID*	Drainage Area (acres)	Runoff Curve Number	Time Of Concentration (minutes)	10-year Runoff (cfs) [Design Storm]	100 year Runoff (cfs)
EX1	7.34	86	14	28.3	45.5
EX2	3.15	83	4	12.9	21.3
EX3	7.96	89	21	27.9	44.3
EX4	5.16	92	12	22.5	34.7
EX5	1.56	80	13	5.3	9.0
EX6	0.36	91	13	1.5	2.3
N1	3.18	98	4	16.0	24.0
N2	3.27	98	13	14.7	22.1
N3	3.47	98	15	15.1	22.7
N4	2.06	98	20	8.3	12.5

* - Sub Basins are shown on Figure 1 – Pre-Development Drainage Map and Figure 2 – Post-Development Drainage Map located in Appendix A – Hydrologic Calculations.

2.2.2 Hydraulic Analysis

2.2.2.1 Pre-Development Conditions

EA performed a hydraulic analysis for the storm drain utilizing the peak runoffs determined in the hydrologic analysis for the pre-development and post-development conditions. The analysis on the pre-development conditions was completed to determine the capacity and effectiveness of the existing system. The analysis confirmed the existing storm drain system has little capacity and the systems often surcharges through the inlets (as observed by property owners in the area) causing the inundation of the roadway sections.

Because of the majority of the runoff being conveyed on the ground surface (or roadway), a hydraulic analysis was performed using the U.S. Army Corps of Engineers Hydrologic Engineering Center Riverine Analysis System (HEC-RAS) program to determine the limit of flooding. Three models were established using the required 10-year 24-hour design storm along with three tidal scenarios [Mean High Water (1.17 NAVD88), Mean Sea Level (-0.25 NAVD88) and Mean Low Water (-1.69 NAVD88)] as tailwater conditions. The input, assumptions and results for the pre-development hydraulic model are included in *Appendix E – Hydraulic Calculations*. The limits of inundation for the existing conditions and tidal scenarios are shown on *Figures 1-3 – Area of Inundation Figures* in *Appendix E – Hydraulic Calculations*.

2.2.2.2 Post-Development Conditions

Due to the proximity to the river and relatively flat existing grades, the storm drain must be constructed at a shallow depth, therefore restricting the size of pipe that can be installed. The largest pipe diameter that can be installed and maintain adequate ground cover is 36-in. Utilizing the results of the post-development hydrologic analysis it was determined that three 36-in. reinforced concrete pipes with an approximate length of 395 linear feet (LF) will be required upstream of the OSPW to convey the peak runoff from the 10-year, 24-hour design storm. The use of equivalent elliptical reinforced concrete pipes was considered as an alternative to circular pipe to reduce the depth of excavation; however, this option was eliminated due to the greater live load from AWI equipment over the wider horizontal pipe section and the reduction in effective lateral support due to the smaller vertical dimension of the elliptical pipe section.

The steel pipes to be installed as part of the OSPW construction will extend approximately 5 ft beyond the limits of the most upgradient OSPW structures (the deadmen). This allows a flanged connection point to extend the steel pipes upstream to connect with Storm Drain Junction 1. Storm Drain Junction Box 1 consists of a concrete vault with the three influent concrete pipes and the three effluent steel pipes, which will provide for the connection of both the concrete storm drains and the steel pipes. This arrangement will negate the need to “mate” two different pipe materials. Based on operational equipment currently used by AWI, the proposed storm drain will consist of a high class reinforced concrete pipe material to withstand future equipment loading. The relocated storm drain will transition to two 36-in. reinforced concrete pipes with an approximate length of 200 LF upstream of a second vault (Storm Drain Junction 2, *Figure 3 – Proposed Condition Plan*) where the system’s design drainage area is smaller in size.

Based on the peak flows for the existing drainage areas, the design also proposes to increase the capacity of the existing storm drain system near the Elm Avenue and Veneer Road intersection by replacing the existing 12-in. and 15-in. storm drains with 24-in. and twin 36-in. reinforced concrete pipes (Storm Drain Junction 4, *Figure 3 – Proposed Condition Plan*). By replacing the undersized pipes and storm drain inlets located in the vicinity of the Elm Avenue and Veneer Road intersection with larger storm drain pipes, the flooding conditions currently experienced during frequent storm events will be reduced and will drain much faster than current conditions. Design of the Elm Avenue storm drain relocation is in accordance with City of Portsmouth and Hampton Roads Regional Construction Standards.

The post-development hydraulic analysis of the storm drain was performed using existing topographic survey and the proposed final grading for the dredged containment area; the proposed final grades are shown in *Figure 2 – Final Grading Plan*. The post-development hydraulic analysis utilized three tidal scenarios for underwater discharge [Mean High Water (1.17 NAVD88), Mean Sea Level (-0.25 NAVD88) and Mean Low Water (-1.69 NAVD88)]. Also, because the runoff from the existing sub basins is primarily conveyed overland, the proposed storm drain junction boxes will provide area inlets that will capture and convey flows. The area inlets will consist of frame and grates cast into Storm Drain Junction Boxes 3 and 4, which will accept overland flow conveyed by Elm Avenue and Veneer Road. The input, assumptions and results for the post-development hydraulic model are included in *Appendix E – Hydraulic Calculations*.

The hydraulic analysis shows that three 36-inch RCP will convey the 10-year, 24-hour design storm during all tidal scenarios without exceeding the rim elevation of the storm drain junctions. By providing additional capacity, the storm drain will convey stormwater runoff more effectively than the existing condition, which currently overtops and floods the roadway during the 10-year, 24-hour and more frequent storms. The analysis also determined the storm drain can convey flows from storm events greater than the 10-year, 24-hour storm. However, during large storm events, rise in tidal elevations in the river due to storm surge may reduce the storm drain system's ability to drain until the storm surge passes.

Calculations associated with the hydraulic analyses are included in Appendix E.

2.2.3 Software Used

The following software was employed in the hydrologic and hydraulic analysis:

- AutoCAD Civil 3D 2012, distributed by Autodesk, was used in the drafting, electronic topographic model, and volume calculations for the project.
- Storm and Sanitary Analysis software, distributed by Autodesk, was utilized to determine stormwater runoff and flow quantities. Storm and Sanitary Analysis software includes TR-55 hydrologic methodology which is a single-event rainfall-runoff hydrologic model for small watersheds developed by the Natural Resources Conservation Service. The model generates hydrographs from both urban and agricultural areas and at selected points along the stream system.. Multiple sub-areas can be modeled within the watershed. Storm and Sanitary Analysis was also utilized for the hydraulic modeling. The software utilizes hydrodynamic routing methods to route runoff through the drainage system. The software can simultaneously simulate dual drainage networks (stormwater sewer network and city streets as separate but connected conveyance pathways) and inlet capacity.
- U.S. Army Corps of Engineers Hydrologic Engineering Center Riverine Analysis System (HEC-RAS) and Autodesk Storm and Sanitary Analysis 2012 were utilized for the hydraulic modeling.

2.2.4 New Area Stormwater Management

City of Portsmouth regulations state that stormwater management is required in the event of increased runoff as a result of development. The construction of the OSPW and East Side Containment Berm will ultimately create a containment facility and produce new land, which will generate increased stormwater runoff.

Due to the nature and proximity of the Elizabeth River (a tidal water body), Virginia Code 10.1 provides an exemption for stormwater quantity management since the project site has a direct discharge to the tidally-influenced receiving water. This eliminates the need to place stormwater quantity management facilities to attenuate post-development flows for discharge at pre-development rates. However, the additional storm runoff from the newly-created land area will require stormwater quality management prior to discharge into the Southern Branch of the Elizabeth River.

In order to collect stormwater runoff from this area for quality treatment, the future grading of the dredged material containment area must slope away from the sheet pile wall toward the west where stormwater quality management facilities are proposed. To provide quality management, the design proposes the use of storm filter vaults (connected at various locations to the new storm drain) to reduce particulate pollutants based on Virginia regulations. **The general locations of the stormwater quality management facilities are shown conceptually on the Design Drawings, and will be more precisely located and designed as part of a future RD effort and are not part of the Elm Avenue storm drain relocation project.** Each property owner will be responsible to maintain their individual stormwater quality management facilities located on their property. EPA expects that the City of Portsmouth will be responsible for maintaining the other components of the relocated storm drain system as shown on the drawings from the Elm Avenue area to the proposed discharge point in the Elizabeth River.

2.2.5 Management of Stormwater Runoff By Property

The overall intent of stormwater management for the newly-created land resulting from the consolidation of dredged Elizabeth River sediments is to prevent runoff on each of the four future properties from crossing property boundaries. Stormwater will be managed on each property individually and separately from the other properties.

Additionally, the grading for the newly-created land as shown on *Figure 2 – Final Grading Plan* is intended to be the final grading configuration for the cap of the dredged material containment facility. Changes to the grading in subsequent design phases or future related remedial designs will require reassessment of drainage and stormwater management requirements.

This section describes how stormwater will be handled on each of the properties once the dredged material containment facility is completely developed. The grading will be performed as part of a future RD effort and is not part of the Elm Avenue storm drain relocation project. Note that these plans could change depending on future discussions with land owners.

<u>AWI Property</u>

Proposed ground surface elevations on future AWI property generated by the completed containment cell will provide a large flat area adjacent to the pile cap. Near the transition from the OSPW pile cap to the southern bulkhead, the ground will slope to the west at approximately 3 percent toward two proposed stormwater quality management facilities, one located along the northern boundary with the FIGG property and one located along the berm to the north of the restored wetland. These facilities will be connected to the relocated Elm Avenue storm drain system via Storm Drain Junction Box 1 and Junction Box 2 for discharge through the OSPW. **The stormwater quality management facilities will be designed as part of a future RD effort and are not part of the Elm Avenue storm drain relocation project.**

FIGG Property
Proposed ground surface elevations on the future FIGG property generated by the completed containment cell will provide a flat area near the OSPW pile cap with a shallow slope to the west before transitioning to a 3 percent slope graded from east to west toward one proposed stormwater quality management facility to be located on the property. This facility will be connected to the proposed storm drain system via Junction Box 2 for discharge through the OSPW. **The stormwater quality management facilities will be designed as part of a future RD effort and are not part of the Elm Avenue storm drain relocation project.**

3975 Elm Avenue Property
Similar to the other properties, the proposed ground surface elevations on the future 3975 Elm Avenue property generated by the completed containment cell will provide a flat area with a shallow slope to the west transitioning to a 3 percent slope graded from east to west toward one proposed stormwater quality management facility to be located near the property. The facility will be connected to the proposed storm drain system via the standard curb inlet being replaced along the east side of Veneer Road for discharge through the OSPW. **The stormwater quality management facilities will be designed as part of a future RD effort and are not part of the Elm Avenue storm drain relocation project.**

PER Property
Proposed ground surface elevations on the future PER property generated by the completed containment cell will have a shallow slope graded to the northwest toward the existing swale constructed as part of the East Side Containment Berm project. It is understood as part of the PER property development, PER will be responsible for capturing surface runoff from the newly-created land and provide stormwater quality management within the PER property boundary prior to discharge into the Southern Branch of the Elizabeth River. Runoff from the PER property will not flow into the relocated Elm Avenue storm drain.

2.2.6 Property Owner Future Land Access

AWI Property
Access to the future AWI property created by the consolidation of dredged sediments will be provided between the FIGG property boundary and proposed Storm Drain Junction Box 1. This access is approximately 240 ft wide.

FIGG Property
Access to the future FIGG property created by the consolidation of dredged sediments will be provided from Elm Avenue.

3975 Elm Avenue Property
Access to the additional property created by the consolidation of dredged sediments will be provided via a transitional ramp located on the property near the western end of the East Side Containment Berm. The ramp will be capable of supporting large truck traffic and is intended to traverse up and over the East Side Containment Berm onto the newly created land. The ramp is not part of the Elm Avenue storm drain relocation design, but will be part of a future RD effort.

PER Property

Access to the newly created PER property will be provided from the existing PER property to the north. Future development by the property owner may warrant additional stormwater management system modifications, which will be the responsibility of the PER property owner.

2.3 GROUNDWATER COLLECTION

Groundwater under the east side of the AWI property generally flows eastward towards the Elizabeth River. Once the OSPW is constructed, groundwater flow to the east will be restricted and groundwater will gradually mound behind the OSPW. The OSPW is designed to support a maximum groundwater elevation of +3.5 ft NAVD88 behind the OSPW. A Groundwater Treatment Alternatives Analysis was prepared by EA (2011) to evaluate the potential mounding of groundwater behind the OSPW and develop alternatives to hydraulically control the mound and treat the groundwater, if needed. The Alternatives Analysis concluded that a groundwater control component is necessary for interception and hydraulic control of contaminated groundwater.

A groundwater collection trench component is included with the design and installation of the Elm Avenue storm drain relocation to provide cost and time savings for EPA. This inclusion provides an increase in construction efficiency by combining two separate systems in the same vicinity into one construction contract. The groundwater collection trench will provide future hydraulic control of groundwater on both the east side of the existing AWI property and the completed dredged material containment cell following its ultimate development. This RA will not result in discharge of groundwater to the Elizabeth River, but the RD does evaluate the potential to discharge in the future. The proximity of the groundwater trench to the storm drain trench is shown on *Figure 3 – Proposed Conditions Plan*, and *Figure 4 – Storm Drain and Groundwater Collection Trench Cross Section* provides a section view of both trenches.

The Elm Avenue Storm Drain Relocation and Groundwater Collection Trench design includes a groundwater collection trench (and monitoring port) for hydraulic control of future groundwater flows, but does not include groundwater treatment. However, the need for groundwater treatment and potential groundwater treatment technologies were evaluated as part of this design and will be further refined in a future RD effort. A memorandum documenting the potential groundwater treatment is included in Appendix F. This design allows space on the site at the end of the proposed groundwater collection trench for the inclusion of a potential future groundwater treatment system. All of these design elements are described in more detail in this section of the report.

The design criteria for the groundwater collection trench include the following:

- Construction of a groundwater collection trench that would maintain the groundwater elevation to a maximum of +3.5 ft at the face of the OSPW, an elevation specified by the wall design as a design constraint.
- A groundwater conveyance system that would convey the water toward the Elizabeth River or a potential treatment system in the future.
- There will be no discharge from the groundwater collection trench at this time.

- A monitoring port will be included in the groundwater trench design.

2.3.1 Design

In order to limit groundwater recharge from the ultimate development of the AWI site, this design assumes a low-permeability material will be placed over the consolidated dredged material. The groundwater model used in this design analysis assumes no recharge to the substrate under the cap following future site development. As a result, the runoff that would drain toward the west and the relocated storm drain is maximized. The specific cap material will be determined as part of a future RD effort but it is anticipated to be a tightly packed granular material (e.g., CR-6). This material has very low permeability, a high load-bearing capacity, and is similar to the majority of the existing AWI property surface.

Details of the groundwater collection trench design are discussed in this section, including:

- Placement
- Sizing and Elevation
- Grading/Excavation/Material
- Monitoring and Potential Future Treatment

2.3.1.1 Placement

A 2-ft-wide by 3-ft-high gravel-filled trench is proposed to the west of the proposed relocated storm drain such that its orientation is roughly perpendicular (north-south) to the anticipated groundwater flow. The groundwater models that were performed as part of this design determined that the construction of one groundwater collection trench located on the west side of the storm drain trench would provide sufficient hydraulic control of the groundwater. This layout will allow the trench to intercept the groundwater flowing from both the east and west, acting as a path of higher hydraulic conductivity. The required trench length is approximately 450 LF. The trench will be constructed such that it will allow water to drain by gravity toward storm drain Junction Box 1 for future discharge through the OSPW to the Elizabeth River.

2.3.1.2 Sizing and Elevation

The groundwater collection trench is sized to capture groundwater from all portions of the AWI property including the area west of Burton’s Point Road and behind the OSPW (from the future consolidated dredged material containment area). Design constraints for the OSPW require that the groundwater elevation in the consolidated dredged material be limited to an elevation of +3.5 ft (NAVD88). In order to determine the appropriate size and elevation of the groundwater collection trench, a hydrogeologic model was utilized.

The hydrogeologic model described in the Groundwater Treatment Alternatives Analysis (EA 2011) was used to model multiple groundwater trench layouts with the stormwater design conditions to determine the most viable control design for the groundwater collection trench. The model input data include the OSPW, a low-permeability cap over the dredged material behind the wall, a cap over the remaining AWI property, and trees along portions of the AWI property to promote additional hydraulic control. The model shows that without additional hydraulic control, groundwater is anticipated to mound to an elevation greater than +3.5 ft (NAVD88) on the west side of the OSPW. The model also illustrated that additional trees located on the PER or FIGG properties may be beneficial in providing further hydraulic control of groundwater. This evaluation of planting additional trees will be completed by groundwater modeling during a future RD effort.

A 2-ft by 3-ft groundwater trench was modeled to assess its ability to decrease the groundwater elevation at the OSPW and to aid in hydraulic control of groundwater. Different alignments and elevations were evaluated for their effect on groundwater levels and flow directions. The relocated Elm Avenue storm drain, which will have an impermeable liner in the excavated trench, was also added to the model input as a zone of decreased conductivity to assess whether it may impede groundwater flow in the area of the groundwater collection trench. The model shows the lined storm drain trench has only minimal effect on groundwater flow and therefore is not expected to interfere with groundwater collection. The model indicated that a trench with a bottom elevation +1.5 ft, oriented from northwest to southeast and located parallel to and just west of the relocated storm drain, would limit the groundwater elevation at the face of the OSPW to 3 ft, below the maximum elevation of +3.5 ft at the OSPW. A trench at elevation +1.5 ft also creates a hydraulic gradient that promotes water flow into the trench. The gradient enables the trench to collect groundwater from an area of approximately 12 acres, including much of the area under the dredged material cap, as well as adjacent areas to the west of the trench (See *Figure 3 – Groundwater Flow Directions for Groundwater Trench at Elev. +1.5,* Appendix F). The model indicates that the trench collects water from these areas, with no water passing under the collection trench (See *Figure 4 – Groundwater Flow Cross-Section for Groundwater Trench at Elev. +1.5,* Appendix F). The modeled rate of flow of groundwater into the trench is 137 cubic feet per day (cfd) (0.7 gallons per minute [gpm]).

For comparison with the trench at +1.5 ft, a trench at 0 ft elevation was also modeled. As expected, the lower elevation trench collects more water from a larger area. The modeled rate of groundwater flow into the trench at 0 ft elevation is 300 cfd (1.6 gpm), collected from an area of approximately 20 acres (See *Figure 5 – Groundwater Flow Directions for Groundwater Trench at Elev. 0,* Appendix F). As with placing the trench at +1.5 ft elevation, the model did not show water flowing under the trench at 0 ft (See *Figure 6 – Groundwater Flow Cross-Section for Groundwater Trench at Elev. 0,* Appendix F). Therefore, it was concluded that a trench with a bottom at 0 ft elevation (combined with capping and trees for additional hydraulic control) would effectively control groundwater flow on the east side of the AWI site, while also limiting water levels along the OSPW to within the design criterion. It also allows for additional elevation drop for future treatment and discharge to the Elizabeth River. This arrangement is included in the design.

To aid in the conveyance of groundwater, the last 20 ft of the groundwater collection trench located at the southeast end of the trench profile contains an 18-in. diameter perforated SDR11 high density polyethylene (HDPE) pipe. Due to the potential for large cranes to cross the groundwater collection trench and damage the 18-in. perforated SDR11 HDPE pipe within the trench, only the outlet section of the groundwater collection trench contains an 18-in. perforated SDR11 HDPE pipe; the last 20 ft is expected to be out of the crane travel path, reducing the potential for crushing. At the southeast end of the trench, the 18-in. perforated SDR11 HDPE pipe transitions to solid SDR11 HDPE. The solid pipe will convey groundwater from the trench to a potential future treatment system and the river.

2.3.1.3 Grading/Excavation/Material

The grading and excavation activities associated with the groundwater collection trench design include the following requirements:

- Excavation for the construction of the groundwater collection trench.
- Dewatering and potential treatment of stormwater runoff, groundwater infiltration or any potential ponding areas created by the construction of the groundwater collection trench. Handling, treatment and discharge of stormwater and groundwater from dewatering operation is described in the Contract Specifications.
- Decontamination of all earthmoving equipment and personnel will be decontaminated prior to leaving the project site. Intrusive activities will require the decontamination of all equipment prior to entering public roadways adjacent to site.

The materials of construction for the groundwater collection trench design include the following:

a. Stone

   Clean, washed No. 57 stone meeting the requirements of the Contract Specifications will be used for the groundwater collection trench construction, and will be wrapped in 16-ounce nonwoven geotextile fabric to prevent sediment and debris from entering the stone-filled trench.

b. HDPE Pipe

   The outlet piping will consist of 20 ft of perforated SDR11 18-in. HDPE and will transition to 6 ft of solid 18-inch HDPE pipe (including the coupler) outside of the groundwater collection trench.

c. Monitor Port

   The solid HDPE pipe will have a 6-in. HDPE tee which will extend above grade as a monitoring port. The monitoring port will be protected with a steel locking casing that will be set within a 2-ft by 2-ft concrete pad.

d. Backfill and Grading Material

   Remaining material needed for the construction of the groundwater collection trench will be clean, imported borrow material meeting criteria required by the specifications for this project.

2.3.1.4 Monitoring and Potential Future Treatment

Volatile organic compounds (VOCs), metals, PAHs, SVOCs and other constituents have been detected in groundwater at the site. Existing groundwater concentrations for some compounds exceed the Virginia water quality standards and cannot be discharged to the river without treatment or adequate mixing zones. Additionally, groundwater which mounds in the area of the future dredged material containment cell may come into contact with contaminated dredged material and may have concentrations of VOCs and metals even higher than existing groundwater. Potential treatment of groundwater and the potential application of mixing zones were evaluated such that groundwater could be discharged in the future to the Elizabeth River in compliance with the Virginia water quality standards.

Conceptual design of groundwater treatment vaults assuming a passive treatment media was performed based on historical data collected at various monitoring wells at the site. Upon evaluation, it was determined that additional groundwater quality information would be needed to properly size treatment structures. Further evaluation of future analytical data is required to determine residence time requirements and type of treatment media necessary to treat the groundwater for compliance with Virginia water quality standards prior to discharge to the river. Preliminary design, including calculations and conceptual layouts are included in Appendix F.

In addition to water quality standards for discharge, Virginia also permits the use of mixing zones for certain discharges. A mixing zone would allow for higher concentrations to be discharged to the Elizabeth River based on the velocity of the discharge and the corresponding calculated dilution factor. The use of mixing zones may allow for the reduction or elimination of treatment requirements. Future evaluation based on future groundwater quality data will be required to determine more accurate groundwater concentrations to be discharged from the groundwater collection trench and whether discharge would need to be pumped to the river in order to increase the discharge velocity and corresponding dilution factors. Further discussion regarding mixing zones and preliminary mixing zone modeling and calculations are included in Appendix F.

To monitor the quality of groundwater collecting in the groundwater trench, the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench design includes a 6-in. HDPE at the end of the groundwater collection trench that will serve as a monitoring port. For this design, the solid 18-in. HDPE pipe will be capped just beyond the monitoring port. **Since the pipe is capped, there will be no groundwater treatment or discharge to the Elizabeth River after completion of this phase of the RA.**

The intent of the monitoring port will be to sample groundwater from the collection trench to collect groundwater data during and following the placement of dredged material. The analytical data from the samples collected will be used to determine more accurate baseline contaminant concentrations. Based on this data, future treatment facilities may be added downstream of the trench which will allow the groundwater to flow from the trench, through the treatment system, and to a discharge point into the Elizabeth River. It is also possible that a refined mixing zone analysis will indicate that treatment is not required. **Groundwater collection and treatment facilities may be added as part of a future RD effort and are not part of the Elm Avenue storm drain relocation project.**

2.4 GEOTECHNICAL INVESTIGATION

Due to the size of the storm drain pipes, the expected live loads following construction, and the fact that a portion of the pipeline route will be built on fill, a geotechnical assessment of the existing site soils was conducted. EA contracted Schnabel Engineering, LLC to collect geotechnical borings along the route of the new storm drain alignment and perform geotechnical analyses for confirmation that the site soils can properly support the new pipeline and the junction boxes.

The geotechnical analysis included subsurface exploration, soil laboratory tests, and development of geotechnical engineering recommendations, including:

- Nature of Existing Soils
- Excavation Support Recommendations
- Dewatering Methods Recommendations
- Subgrade Preparation Recommendations
- Backfill/Bedding/Junction Box Materials Recommendations
- Potential for Fill Settlement

The geotechnical report, including the geotechnical boring logs, is included in Appendix G.

To assist in this effort, EA utilized existing information to the extent possible, requesting existing boring data from FIGG, who collected geotechnical information as part of their bridge design in the vicinity of the storm drain relocation. Additionally, EA evaluated the effect of the future industrial site use by AWI, including the effect of the large travel cranes on the pipeline. This information was used to refine the design for the pipe, the evaluation of the pipe loads, the selection of the geosynthetics, backfill and pipe bedding, and the design of the junction boxes.

2.5 DENSE NON-AQUEOUS PHASE LIQUIDS (DNAPL)

Previous field investigations were conducted by CDM to assess the subsurface geology and determine the lateral extent of surface and subsurface DNAPL in the area of the Elm Avenue storm drain relocation. These investigations revealed existing subsurface DNAPL contamination throughout the eastern half of the AWI property. To minimize the potential for migration of these contaminants, the Elm Avenue storm drain trench will be lined with an HDPE membrane liner. Excavated soil is considered contaminated and will be handled in accordance with the requirements set forth in the specifications and transported to existing Stockpile Area A on the west side of the AWI site. DNAPL, if encountered in soil, will be excavated and deposited in Stockpile Area A. If DNAPL is encountered in the water from the dewatering operation, it will be removed by appropriate treatment media. .

Available soil characteristics and water quality data in the vicinity of the proposed storm drain relocation is provided in Attachment 2 of the Contract Specifications.

2.6 EROSION AND SEDIMENT CONTROL

In accordance with the *Virginia Erosion and Sediment Control Regulations* and *City of Portsmouth Erosion and Sediment Control Ordinance* regulations, localized flooding, offsite migration of sediment, and stream channel erosion of the existing waterways will be controlled during all land-disturbing activities through implementation of sediment control devices, methods, and installation procedures set forth by the regulations. A sequence for the establishment of the erosion and sediment controls is provided on the plans in order to describe how and when the controls should be installed and removed in relationship to construction activities.

Erosion and sediment control for the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench construction includes two stabilized construction entrances for construction access and silt fence and temporary diversion dikes for perimeter control. Silt fence will be used along Elm Avenue to provide sediment control and to direct construction traffic to the stabilized construction entrances. The Contractor will be responsible for providing access to the river for AWI at all times during construction. Temporary diversion dikes were chosen in lieu of silt fence because the dikes can be relocated more easily than the fencing should AWI equipment need to traverse the area during construction. This will provide more flexibility for the Contractor to provide erosion control around work areas while providing AWI access across the project site. Temporary diversion dikes will provide clean water diversion and convey offsite water to rock outlet protection areas prior to discharge. Onsite water will be conveyed via the temporary diversion dikes to stone outlet structures where sediment will be trapped by the structure and clean water will be allowed to discharge.

Stormwater bypass of the existing Elm Avenue and Veneer Road storm drains and groundwater dewatering during trench excavation will be required during the construction process. Provisions for the treatment of groundwater and surface water runoff are provided in Specification Sections 01 57 13 - Erosion and Sediment Control and 02 61 13 - Excavation and Handling of Contaminated Materials.

2.7 AMBIENT AIR STANDARDS CALCULATIONS

Air monitoring for both RA and non-RA workers will be conducted during the project when contaminated material is being excavated, handled, or treated. Ambient air standards (risk-based criteria) for non-RA workers have been provided to the Contractor in the Specifications. Ambient air standards were calculated using guidance provided in EPA's Risk Assessment Guidance for Superfund (RAGS) and Regional Screening Levels (RSLs) for Superfund Sites (May 2012). Exposure parameters were taken from the RAGS and RSL guidance, except for site-specific inputs that include target risk and exposure duration. The calculations were performed with Microsoft Excel 2007 spreadsheets developed by EA and are based upon the same equations presented for ambient air inhalation exposure in RAGS and the RSLs.

Ambient air standards were calculated for the soil contaminants of concern (COCs) identified in the ROD (polycyclic aromatic hydrocarbons [PAHs], PCP, arsenic, antimony, iron, and thallium), as well as the BTEX compounds (benzene, toluene, ethylbenzene, and xylenes) due to their presence in groundwater. The target risk was set at 1×10^{-5} since all of the PAHs identified in the EPA RSL Table were included in the calculations. For non-carcinogens, a target of 1.0 was used instead of 0.1 since all of the non-carcinogenic compounds in soil do not have the same target organ. The exposure duration was assumed to be 5 months (the anticipated duration of the Elm Avenue Storm Drain and Groundwater Collection Trench construction activities).

Calculations associated with the ambient air standards are included in Appendix H. Contractor requirements for air monitoring, including minimum requirements for onsite and perimeter monitoring, screening levels based on risk, and trigger concentrations for implementing action are provided in the specifications.

2.8 REGULATORY CONSIDERATIONS

The following regulations were considered in the production of the Elm Avenue Storm Drain and Groundwater Collection Trench design:

Virginia Water Protection General Permit Regulation— A permit will not be required for the stormwater management system; however, the substantive requirements of the permit will be met. In accordance with the regulations, any imported fill material associated with the stormwater management and/or drainage conveyance systems will be clean and free of contaminants in toxic concentrations or amounts in accordance with all applicable laws and regulations.

Virginia Pollutant Abatement (VPA) Permit Regulation—This requirement regulates the stormwater collected from the surface of the site adjacent to the Southern Branch of the Elizabeth River, a state surface water. This discharge is required to comply with the substantive requirements of the VPA permit allowing no point source discharge of pollutants to the surface water except in the case of a storm even greater than the 25-year, 24-hour storm. This remediation is only affecting the discharge point and not the surface conditions of the existing site. ; This remediation has no impact on the requirements AWI may currently have for stormwater discharge.

National Pollutant Discharge Elimination System (NPDES), Virginia Pollutant Discharge Elimination System (VPDES) General Permit Regulation and City of Portsmouth Stormwater Management Ordinance—No administrative permitting or review document submissions are required for the stormwater management system from either the Commonwealth of Virginia or the City of Portsmouth; however, the substantive requirements of both the general permit and ordinance will be met during construction activity. In accordance with the regulations, localized flooding and stream channel erosion of the existing waterways will be controlled by managing the post-development stormwater runoff to the extent practicable and equal to or better than the pre-development runoff conditions.

3.0 DRAWINGS AND SPECIFICATIONS

This section provides the lists of drawings and specifications that comprise the Elm Avenue Storm Drain Relocation and Groundwater Collection Trench RD.

3.1 DESIGN DRAWINGS

The design drawings consist of the following sheets:

Drawing Number	Sheet Number	Sheet Title
T-1	1	Title Sheet
T-2	2	Index Of Drawings/General/ Notes/Abbreviations/Legend
C-1	3	Site Plan/Key Sheet
C-2	4	Existing Conditions Plan
C-3	5	Demolition Plan
C-4	6	Proposed Conditions Plan
C-5	7	Storm Drain Plan and Profile I
C-6	8	Storm Drain Plan and Profile II
C-7	9	Groundwater Collection Trench Plan and Profile
C-8	10	Storm Drain Sections I
C-9	11	Storm Drain Sections II
C-10	12	Storm Drain Sections III
C-11	13	Storm Drain Details I
C-12	14	Storm Drain Details II
C-13	15	Storm Drain & Groundwater Collection Trench Details
C-14	16	Storm Drain Junction Box 1 Details
C-15	17	Storm Drain Junction Box 2 Details
C-16	18	Storm Drain Junction Box 3 Details
C-17	19	Storm Drain Junction Box 4 Details
ES-1	20	Erosion And Sediment Control Plan
ES-2	21	Erosion And Sediment Control Details
ES-3	22	Erosion And Sediment Control Notes

3.2 TECHNICAL SPECIFICATIONS

The following is a list of the specifications for the project.

DIVISION 01 – GENERAL REQUIREMENTS

01	11	00	SUMMARY OF WORK
01	31	00	PROJECT MANAGEMENT AND COORDINATION
01	33	00	SUBMITTAL PROCEDURES
01	35	29.13	HEALTH, SAFETY, AND EMERGENCY RESPONSE PROCEDURES FOR CONTAMINATED SITES
01	35	40	ENVIRONMENTAL MANAGEMENT
01	35	45	CHEMICAL DATA QUALITY CONTROL
01	45	00	QUALITY CONTROL
01	50	00	TEMPORARY CONSTRUCTION FACILITIES AND CONTROLS
01	57	13	EROSION AND SEDIMENT CONTROL
01	77	00	CLOSEOUT PROCEDURES

DIVISION 02 - EXISTING CONDITIONS

02	61	13	EXCAVATION AND HANDLING OF CONTAMINATED MATERIAL

DIVISION 03 - CONCRETE

03	00	00	CONCRETE
03	11	13	CONCRETE FORMWORK
03	15	13	WATERSTOPS
03	21	00	REINFORCING STEEL

DIVISION 09 - FINISHES

09	97	13.26	COATING OF STEEL PIPE

DIVISION 31 – EARTHWORK

31	00	00	EARTHWORK
31	05	19	GEOSYNTHETICS
31	11	00	CLEARING AND GRUBBING
31	23	19	DEWATERING

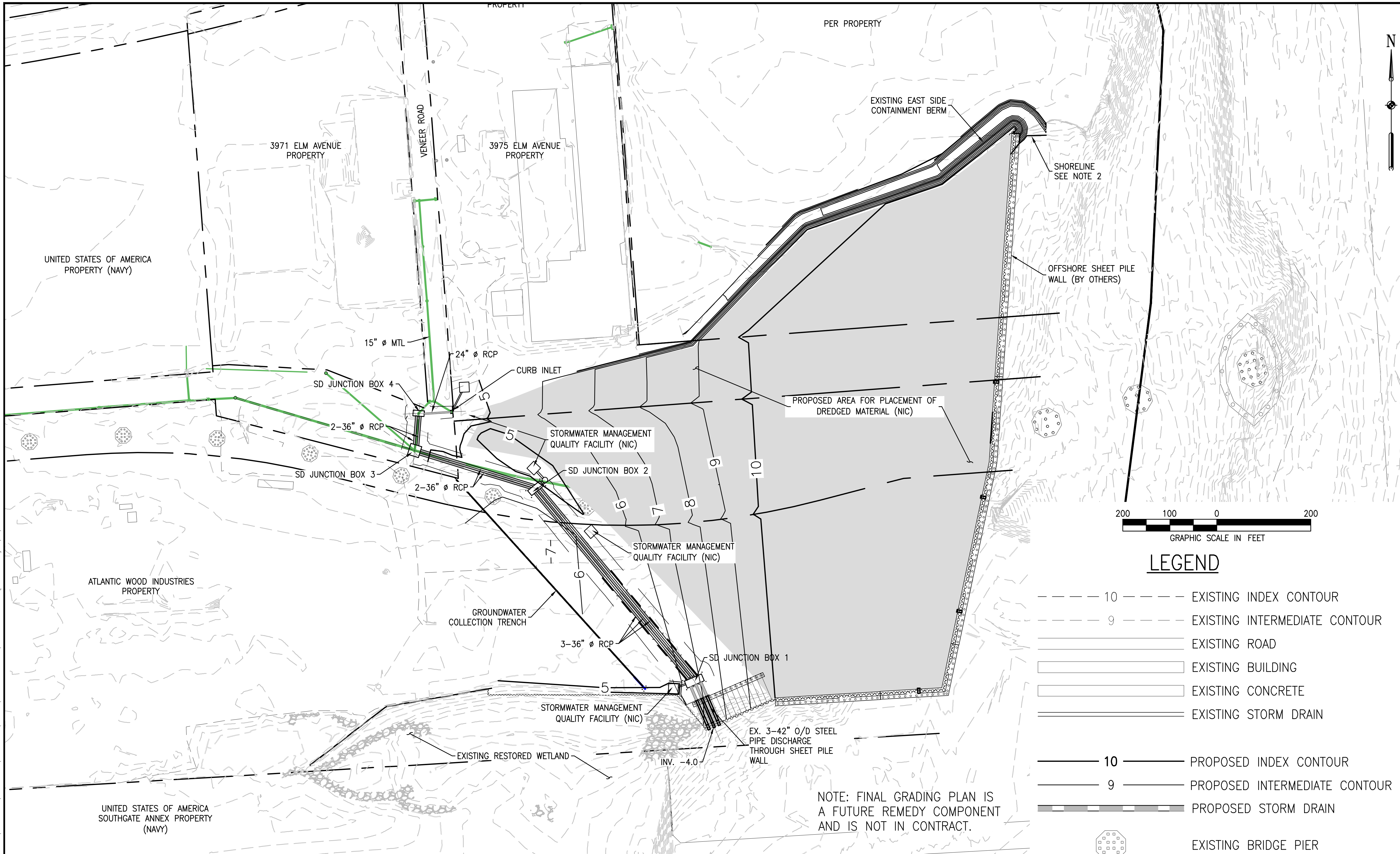
DIVISION 33 – UTILITIES

33	40	00	STORM DRAIN AND GROUNDWATER COLLECTION TRENCH

4.0 CONSTRUCTION SCHEDULE

EA prepared a Remedial Action Work Plan (RAWP) Addendum to define the construction scope and schedule. EA received approval of the RAWP Addendum on 19 June 2012, with award to an RA subcontractor anticipated in mid September 2012. Onsite construction activities are anticipated to begin mid to late October 2012. Construction is anticipated to last approximately 5 months. The anticipated construction schedule is shown on Figure 5.

Figure 1 – Atlantic Wood Industries Superfund Site - Remedy Components(May 2012)
Portsmouth, Virginia



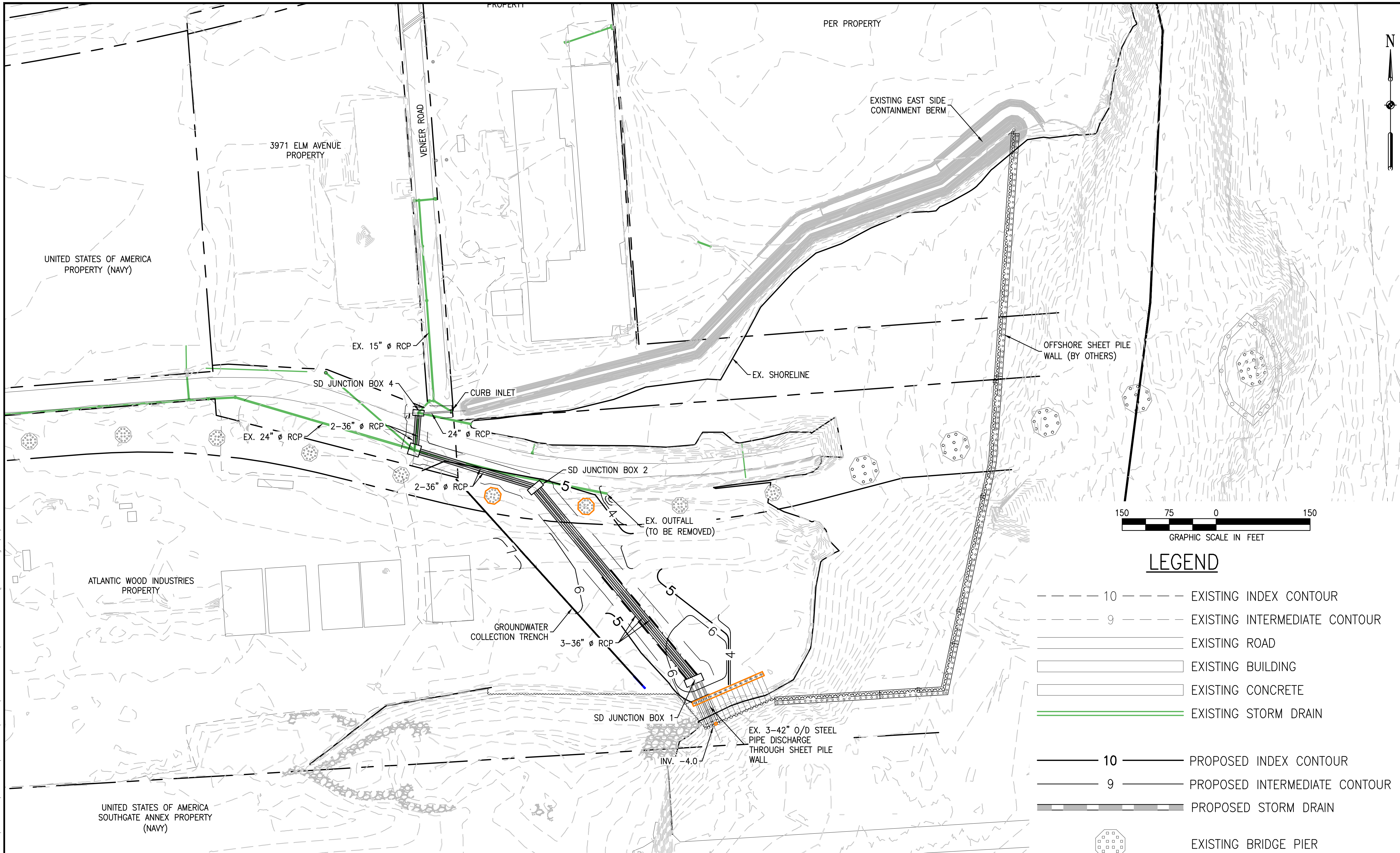
EA ENGINEERING, SCIENCE, AND TECHNOLOGY

DESIGNED BY	DRAWN BY	DATE	PROJECT NO.
JLL	JLL	AUGUST 2012	1453011
CHECKED BY	PROJECT MGR.	DRAWING NO.	FIGURE
GAT	PAP	2	2

REMEDIAL DESIGN
ELM AVEMUE STORM DRAIN RELOCATION AND GROUNDWATER COLLECTION TRENCH
PORTSMOUTH, VIRGINIA

FINAL GRADING PLAN

FILE PATH: L:\NON DOD\AWI CAD\PHASE 2\GW SWM\FINAL BOD FIGURES\FIGURE 2.DWG [FIGURE 2] 8/7/12



FILE PATH: L:\NON DOD\AWI CAD\PHASE 2\GW SWM\FINAL BOD FIGURES\FIGURE 3.DWG [FIGURE 3] 8/7/12

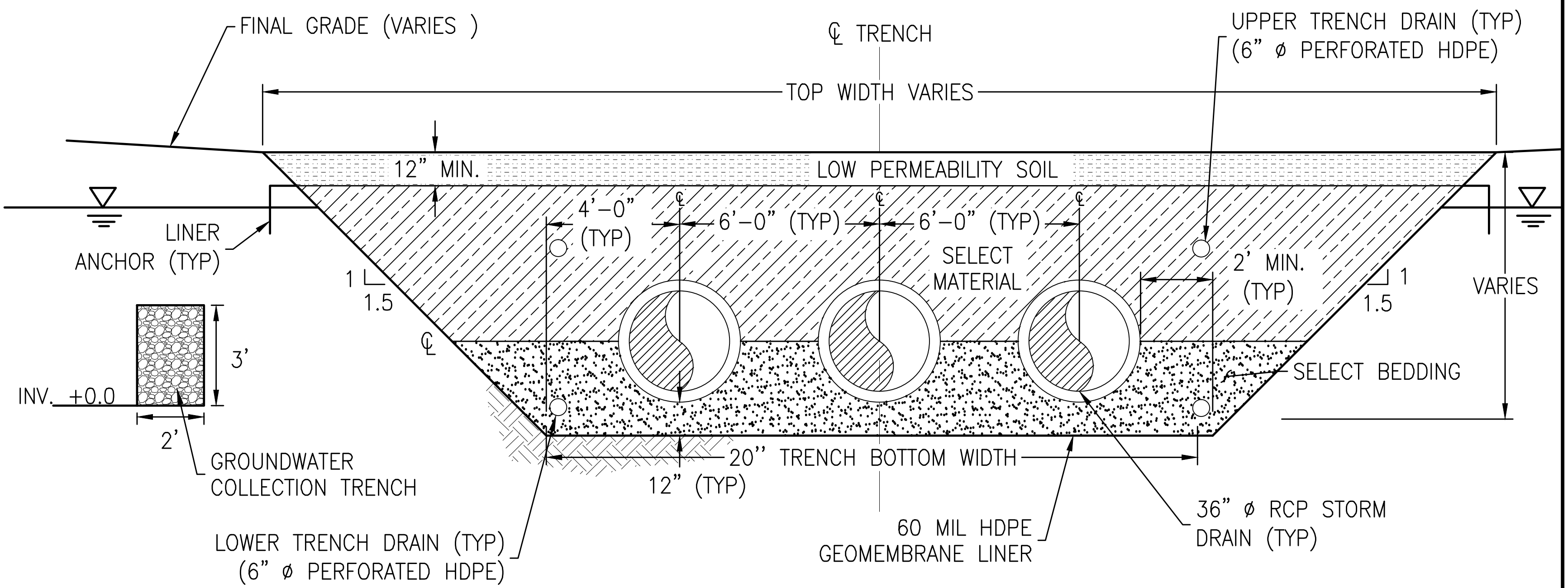
EA ENGINEERING, SCIENCE, AND TECHNOLOGY

DESIGNED BY	DRAWN BY	DATE	PROJECT NO.
JLL	JLL	AUGUST 2012	1453011
CHECKED BY	PROJECT MGR.	DRAWING NO.	FIGURE
GAT	PAP	3	3

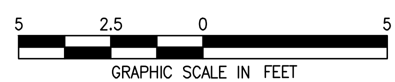
REMEDIAL DESIGN
ELM AVEMUE STORM DRAIN RELOCATION AND
GROUNDWATER COLLECTION TRENCH
PORTSMOUTH, VIRGINIA

PROPOSED CONDITIONS PLAN

FILE PATH: L:\NON DOD\AWI CAD\PHASE 2\GW SWM\FINAL\BOD FIGURES\FIGURE 4.DWG [LAYOUT1] 8/7/12



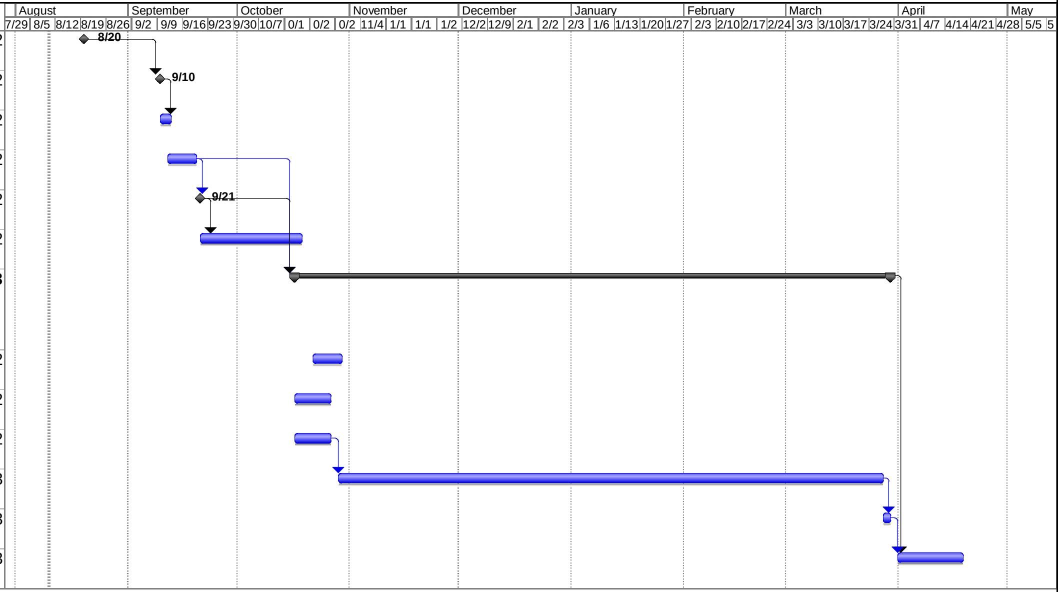
NOTE: PORTION OF STORM DRAIN CONSISTS OF 2–36" ø RCP.



EA ENGINEERING, SCIENCE, AND TECHNOLOGY	ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER COLLECTION TRENCH PORTSMOUTH, VIRGINIA	STORM DRAIN AND GROUNDWATER COLLECTION TRENCH CROSS SECTION	DESIGNED BY JLL	DRAWN BY JLL	DATE AUGUST 2012	PROJECT NO. 1453011
			CHECKED BY GAT	PROJECT MGR. PAP	SCALE 1"=5'	FIGURE 4

Figure 5
Construction Schedule
Elm Avenue Storm Drain Relocation and Groundwater Collection Trench
Atlantic Wood Industries Superfund Site
Portsmouth, Virginia

ID	Task Name	Duration	Start	Finish
1	Pre Bid Site Visit	0 days	Mon 8/20/12	Mon 8/20/12
2	Bids Received	0 days	Mon 9/10/12	Mon 9/10/12
3	Review Bids	3 days	Mon 9/10/12	Wed 9/12/12
4	Subcontract Award to RA Subcontractor	6 days	Wed 9/12/12	Wed 9/19/12
5	Pre-Construction Meeting	0 days	Fri 9/21/12	Fri 9/21/12
6	Pre-Construction Submittals	20 days	Fri 9/21/12	Thu 10/18/12
7	**Elm Avenue Storm Drain Relocation and Groundwater Collection Trench Construction**	**118 days**	**Wed 10/17/12**	**Fri 3/29/13**
8	Mobilization	6 days	Mon 10/22/12	Mon 10/29/12
9	Utility Location	8 days	Wed 10/17/12	Fri 10/26/12
10	Install Erosion and Sediment Controls	8 days	Wed 10/17/12	Fri 10/26/12
11	Storm Drain and Groundwater Collection Trench Construction	108 days	Mon 10/29/12	Wed 3/27/13
12	Demobilization	2 days	Thu 3/28/13	Fri 3/29/13
13	Contract Closeout	14 days	Mon 4/1/13	Thu 4/18/13



Project: AWI - Phase 2A
Date: Fri 8/10/12

Task · Split · Progress · Milestone · Summary · Project Summary · External Tasks · External Milestone · Deadline

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Appendix A

Hydrologic Calculations

- **Hydrologic Calculations - Pre-Development**
- **Pre-Development Drainage Area Map**
- **Hydrologic Calculations - Post-Development**
- **Post-Development Drainage Area Map**
- **Hydrologic Summary Table**

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Hydrologic Calculations
Pre-Development

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Prepared by JL
Checked by GAT

Subbasin Hydrology

Subbasin : EX1

Input Data

Area (ac) 7.34
Weighted Curve Number 86.26
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	0.45	C	98.00
Gravel roads	5.27	C	89.00
> 75% grass cover, Good	1.61	C	74.00
Composite Area & Weighted CN	7.33		86.26

Time of Concentration

TOC Method : SCS TR-55

Sheet Flow Equation :

Tc = (0.007 * ((n * Lf)^0.8)) / ((P^0.5) * (Sf^0.4))

Where :

Tc = Time of Concentration (hr)
n = Manning's roughness
Lf = Flow Length (ft)
P = 2 yr, 24 hr Rainfall (inches)
Sf = Slope (ft/ft)

Shallow Concentrated Flow Equation :

V = 16.1345 * (Sf^0.5) (unpaved surface)
V = 20.3282 * (Sf^0.5) (paved surface)
V = 15.0 * (Sf^0.5) (grassed waterway surface)
V = 10.0 * (Sf^0.5) (nearly bare & untilled surface)
V = 9.0 * (Sf^0.5) (cultivated straight rows surface)
V = 7.0 * (Sf^0.5) (short grass pasture surface)
V = 5.0 * (Sf^0.5) (woodland surface)
V = 2.5 * (Sf^0.5) (forest w/heavy litter surface)
Tc = (Lf / V) / (3600 sec/hr)

Where:

Tc = Time of Concentration (hr)
Lf = Flow Length (ft)
V = Velocity (ft/sec)
Sf = Slope (ft/ft)

Channel Flow Equation :

V = (1.49 * (R^(2/3)) * (Sf^0.5)) / n
R = Aq / Wp
Tc = (Lf / V) / (3600 sec/hr)

Where :

Tc = Time of Concentration (hr)
Lf = Flow Length (ft)
R = Hydraulic Radius (ft)
Aq = Flow Area (ft²)
Wp = Wetted Perimeter (ft)
V = Velocity (ft/sec)
Sf = Slope (ft/ft)
n = Manning's roughness

Prepared by JL
Checked by [illegible]

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.011	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	1.14	0.00	0.00
Computed Flow Time (min) :	1.47	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	846	0.00	0.00
Slope (%) :	.5	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.14	0.00	0.00
Computed Flow Time (min) :	12.37	0.00	0.00

Total TOC (min) 13.84

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 4.44
Peak Runoff (cfs) 28.26
Weighted Curve Number 86.26
Time of Concentration (days hh:mm:ss) 0 00:13:50

Elm Avenue Storm Drain Relocation
Atlantic Wood Industries, Inc.

Prepared by JL
Checked by GHT

Subbasin : EX2

Input Data

Area (ac) 3.15
Weighted Curve Number 82.86
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	1.05	C	98.00
Gravel roads	0.18	C	89.00
> 75% grass cover, Good	1.92	C	74.00
Composite Area & Weighted CN	3.15		82.86

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.010	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.3	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	1.36	0.00	0.00
Computed Flow Time (min) :	1.22	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	168	0.00	0.00
Slope (%) :	1.3	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.84	0.00	0.00
Computed Flow Time (min) :	1.52	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.015	0.00	0.00
Flow Length (ft) :	214	0.00	0.00
Channel Slope (%) :	.3	0.00	0.00
Cross Section Area (ft²) :	.88	0.00	0.00
Wetted Perimeter (ft) :	2.6	0.00	0.00
Velocity (ft/sec) :	2.64	0.00	0.00
Computed Flow Time (min) :	1.35	0.00	0.00

Total TOC (min) 4.10

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 4.08
Peak Runoff (cfs) 12.91
Weighted Curve Number 82.86
Time of Concentration (days hh:mm:ss) 0 00:04:06

Elm Avenue Storm Drain Relocation
Atlantic Wood Industries, Inc.

Prepared by JL
Checked by GAT

Subbasin : EX3

Input Data

Area (ac) 7.96
Weighted Curve Number 88.62
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	4.85	C	98.00
> 75% grass cover, Good	3.11	C	74.00
Composite Area & Weighted CN	7.96		88.62

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.011	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	0.86	0.00	0.00
Computed Flow Time (min) :	1.94	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	68	633	351
Slope (%) :	29	.5	.1
Surface Type :	Unpaved	Paved	Unpaved
Velocity (ft/sec) :	8.69	1.44	0.51
Computed Flow Time (min) :	0.13	7.33	11.47

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.015	0.00	0.00
Flow Length (ft) :	167	0.00	0.00
Channel Slope (%) :	.5	0.00	0.00
Cross Section Area (ft²) :	1.5	0.00	0.00
Wetted Perimeter (ft) :	2	0.00	0.00
Velocity (ft/sec) :	5.80	0.00	0.00
Computed Flow Time (min) :	0.48	0.00	0.00

Total TOC (min) 21.34

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 4.69
Peak Runoff (cfs) 27.91
Weighted Curve Number 88.62
Time of Concentration (days hh:mm:ss) 0 00:21:20

Elm Avenue Storm Drain Relocation
Atlantic Wood Industries, Inc.

Prepared by JL
Checked by GAT

Subbasin : EX4

Input Data

Area (ac) 5.16
Weighted Curve Number 92.09
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	3.89	C	98.00
> 75% grass cover, Good	1.27	C	74.00
Composite Area & Weighted CN	5.16		92.09

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	67	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	0.13	0.00	0.00
Computed Flow Time (min) :	8.61	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	79	300	0.00
Slope (%) :	25.5	1	0.00
Surface Type :	Unpaved	Paved	Unpaved
Velocity (ft/sec) :	8.15	2.03	0.00
Computed Flow Time (min) :	0.16	2.46	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.027	0.00	0.00
Flow Length (ft) :	326	0.00	0.00
Channel Slope (%) :	.9	0.00	0.00
Cross Section Area (ft^2) :	2	0.00	0.00
Wetted Perimeter (ft) :	2	0.00	0.00
Velocity (ft/sec) :	5.24	0.00	0.00
Computed Flow Time (min) :	1.04	0.00	0.00

Total TOC (min) 12.27

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 5.08
Peak Runoff (cfs) 22.49
Weighted Curve Number 92.09
Time of Concentration (days hh:mm:ss) 0 00:12:16

Subbasin : EX5

Input Data

Area (ac) 1.56
Weighted Curve Number 80.00
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	0.39	C	98.00
> 75% grass cover, Good	1.17	C	74.00
Composite Area & Weighted CN	1.56		80.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	11.86	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	117	101	0.00
Slope (%) :	17	.5	0.00
Surface Type :	Unpaved	Paved	Unpaved
Velocity (ft/sec) :	6.65	1.44	0.00
Computed Flow Time (min) :	0.29	1.17	0.00

Total TOC (min) 13.33

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 3.78
Peak Runoff (cfs) 5.28
Weighted Curve Number 80.00
Time of Concentration (days hh:mm:ss) 0 00:13:20

Elm Avenue Storm Drain Relocation
Atlantic Wood Industries, Inc.

Prepared by JL
Checked by GAT

Subbasin : EX6

Input Data

Area (ac) 0.36
Weighted Curve Number 90.67
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	0.15	C	98.00
Gravel roads	0.16	C	89.00
> 75% grass cover, Good	0.05	C	74.00
Composite Area & Weighted CN	0.36		90.67

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	0.15	0.00	0.00
Flow Length (ft) :	65	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	0.10	0.00	0.00
Computed Flow Time (min) :	11.09	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	72	0.00	0.00
Slope (%) :	.1	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	0.51	0.00	0.00
Computed Flow Time (min) :	2.35	0.00	0.00

Total TOC (min) 13.44

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 4.92
Peak Runoff (cfs) 1.50
Weighted Curve Number 90.67
Time of Concentration (days hh:mm:ss) 0 00:13:26

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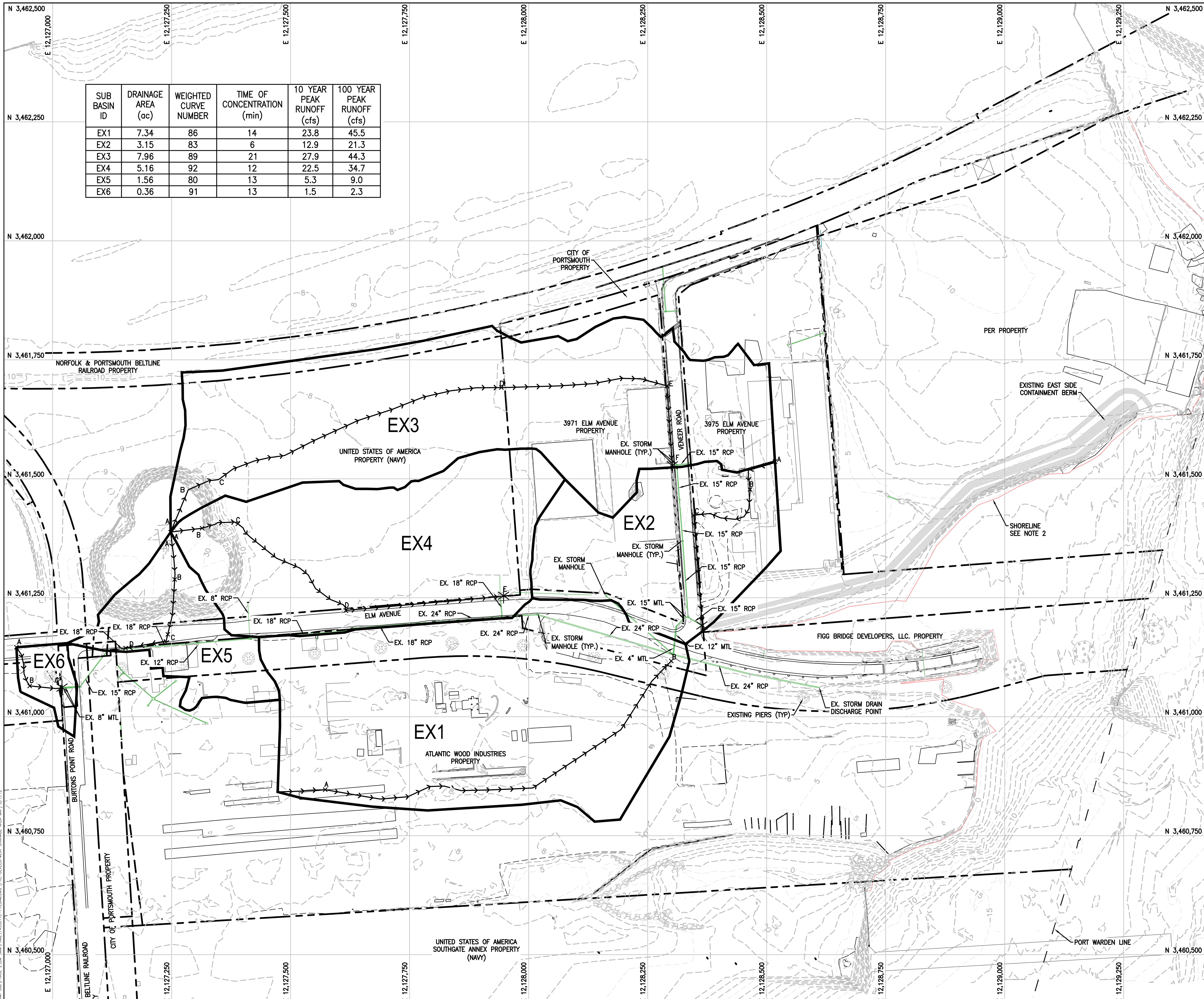
SUB BASIN ID	DRAINAGE AREA (ac)	WEIGHTED CURVE NUMBER	TIME OF CONCENTRATION (min)	10 YEAR PEAK RUNOFF (cfs)	100 YEAR PEAK RUNOFF (cfs)
EX1	7.34	86	14	23.8	45.5
EX2	3.15	83	6	12.9	21.3
EX3	7.96	89	21	27.9	44.3
EX4	5.16	92	12	22.5	34.7
EX5	1.56	80	13	5.3	9.0
EX6	0.36	91	13	1.5	2.3



LEGEND

SYMBOL	DESCRIPTION
	EXISTING INDEX CONTOUR
	EXISTING INTERMEDIATE CONTOUR
	EXISTING BUILDING
	EXISTING ROAD
	EXISTING GRAVEL ROAD
	EXISTING RAILROAD
	EXISTING FENCE
	EXISTING GUARD RAIL
	EXISTING FIRE HYDRANT
	EXISTING STORM DRAIN INLET
	EXISTING MONITORING WELL
	EXISTING MONITORING WELL AND BOLLARDS
	EXISTING BENCHMARK
	EXISTING CONTROL POINT
	EXISTING UTILITY POLE
	EXISTING LIGHT POLE
	EXISTING SIGN
	EXISTING BILLBOARD
	EXISTING MONUMENT
	EXISTING ELECTRIC MANHOLE
	EXISTING ELECTRIC METER
	EXISTING PULL BOX
	EXISTING TELEPHONE MANHOLE
	EXISTING TELEPHONE PEDESTAL
	EXISTING GAS LINE MARKER
	EXISTING WETLAND
	EXISTING RIPRAP
	EXISTING DITCH CENTERLINE
	EXISTING TREE OR BRUSH LINE
SS SS SS SS	EXISTING SANITARY FORCEMAIN
SAN SAN SAN	EXISTING SANITARY SEWER PIPE (GRAVITY)
V V V	EXISTING WATER PIPE
	EXISTING STORM DRAIN
E E E E	EXISTING UNDERGROUND ELECTRIC LINE
UNK UNK UNK	EXISTING UNDERGROUND UNKNOWN UTILITY
O/H O/H O/H	EXISTING OVERHEAD ELECTRIC LINE
UT UT UT UT	EXISTING UNDERGROUND TELEPHONE LINE
OT OT OT OT	EXISTING OVERHEAD TELEPHONE LINE
GAS GAS GAS	EXISTING GAS LINE
STM STM STM	EXISTING STEAM LINE
AIR AIR AIR	EXISTING COMPRESSED AIR LINE
	EXISTING PROPERTY LINE
	EXISTING SHORELINE
	EXISTING BRIDGE PIER
10	PROPOSED INDEX CONTOUR
9	PROPOSED INTERMEDIATE CONTOUR
	PROPOSED STORM DRAIN
	PROPOSED EASEMENT
LOD LOD	LIMIT OF DISTURBANCE
SF SF	SILT FENCE
	TEMPORARY DIVERSION DIKE
	DRAINAGE AREA BOUNDARY
	TIME OF CONCENTRATION PATH

NOTES:

1. CURRENT DEEDS FOR SHORELINE PROPERTIES OWNED BY AWI AND DEEDS FOR THE 3971/3975 ELM AVENUE PROPERTIES (AND LIKELY THE PER PROPERTY, FIGG BRIDGE DEVELOPERS, AND THE U.S. NAVY) STATE THAT THE EASTERN BOUNDARY OF EACH PROPERTY IS THE PORT WARDEN'S LINE. HOWEVER, EPA BELIEVES THAT THE SUB-TIDAL RIVER BOTTOM ADJACENT TO THESE PROPERTIES IS INCLUDED IN A 1711 GRANT FROM THE COLONIAL GOVERNMENT ON BEHALF OF QUEEN ANNE, IN WHICH THE EASTERN BOUNDARY OF THE GRANTED PROPERTY WAS DESCRIBED AS EXTENDING TO THE MIDDLE OF THE RIVER. THE VIRGINIA OFFICE OF THE ATTORNEY GENERAL (VAG) HAS INFORMED EPA THAT PROPERTY OWNERS BEAR THE BURDEN OF PROVING PRIVATE OWNERSHIP OF SUBAQUEOUS BEDS BY GRANT, AS CONTEMPLATED BY VIRGINIA CODE SECTION 28.2-1200. FURTHERMORE, THE VAG CONTENDS THAT, BECAUSE THERE IS NO VIRGINIA CASE LAW FINDING IN FAVOR OF PRIVATE OWNERSHIP OF SUBAQUEOUS BOTTOMLANDS, THE BED OF THE SOUTHERN BRANCH OF THE ELIZABETH RIVER IS COMMONWEALTH PROPERTY.
2. SHORELINE IS APPROXIMATE AND SHOULD BE USED FOR REFERENCE ONLY.

EA ENGINEERING, SCIENCE, AND TECHNOLOGY	DESIGNED BY: JLL	DRAWN BY: JLL	DATE: AUGUST 2012	PROJECT NO.: 1453011	REMEDIAL DESIGN ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER COLLECTION TRENCH ATLANTIC WOOD INDUSTRIES SUPERFUND SITE PORTSMOUTH, VIRGINIA	PRE DEVELOPMENT DRAINAGE MAP
	CHECKED BY: MP	PROJECT MGR.: PAP	DRAWING NO.: DAM1	FIGURE: 1		

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Hydrologic Calculations
Post-Development

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Prepared by JL
Checked by GAT

Subbasin : N1

Input Data

Area (ac) 3.18
Weighted Curve Number 98.00
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	3.18	C	98.00
Composite Area & Weighted CN	3.18		98.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.01	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	1.23	0.00	0.00
Computed Flow Time (min) :	1.36	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.02	0.00	0.00
Flow Length (ft) :	434	0.00	0.00
Channel Slope (%) :	.5	0.00	0.00
Cross Section Area (ft^2) :	3	0.00	0.00
Wetted Perimeter (ft) :	6	0.00	0.00
Velocity (ft/sec) :	3.32	0.00	0.00
Computed Flow Time (min) :	2.18	0.00	0.00

Total TOC (min) 3.54

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 5.76
Peak Runoff (cfs) 15.97
Weighted Curve Number 98.00
Time of Concentration (days hh:mm:ss) 0 00:03:32

Prepared by JL
Checked by GAF

Subbasin : N2

Input Data

Area (ac) 3.27
Weighted Curve Number 98.00
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	3.27	C	98.00
Composite Area & Weighted CN	3.27		98.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.01	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	0.49	0.00	0.00
Computed Flow Time (min) :	3.41	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	229	240	0.00
Slope (%) :	.1	1.87	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	0.51	2.21	0.00
Computed Flow Time (min) :	7.48	1.81	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.018	0.00	0.00
Flow Length (ft) :	105	0.00	0.00
Channel Slope (%) :	.5	0.00	0.00
Cross Section Area (ft²) :	2	0.00	0.00
Wetted Perimeter (ft) :	8	0.00	0.00
Velocity (ft/sec) :	2.32	0.00	0.00
Computed Flow Time (min) :	0.75	0.00	0.00

Total TOC (min) 13.46

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 5.76
Peak Runoff (cfs) 14.72
Weighted Curve Number 98.00
Time of Concentration (days hh:mm:ss) 0 00:13:28

Prepared by JL
Checked by GAT

Subbasin : N3

Input Data

Area (ac)	3.47
Weighted Curve Number	98.00
Rain Gage ID	10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	3.47	C	98.00
Composite Area & Weighted CN	3.47		98.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.01	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	0.49	0.00	0.00
Computed Flow Time (min) :	3.41	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	289	277	0.00
Slope (%) :	.1	1.8	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	0.51	2.16	0.00
Computed Flow Time (min) :	9.44	2.14	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.02	0.00	0.00
Flow Length (ft) :	107	0.00	0.00
Channel Slope (%) :	.5	0.00	0.00
Cross Section Area (ft^2) :	3	0.00	0.00
Wetted Perimeter (ft) :	4	0.00	0.00
Velocity (ft/sec) :	4.35	0.00	0.00
Computed Flow Time (min) :	0.41	0.00	0.00

Total TOC (min) 15.41

Subbasin Runoff Results

Total Rainfall (in)	6.00
Total Runoff (in)	5.76
Peak Runoff (cfs)	15.07
Weighted Curve Number	98.00
Time of Concentration (days hh:mm:ss)	0 00:15:25

Elm Avenue Storm Drain Relocation
Atlantic Wood Industries, Inc.

Prepared by JL
Checked by [signature]

Subbasin : N4

Input Data

Area (ac) 2.06
Weighted Curve Number 98.00
Rain Gage ID .... 10 yr 24 hr

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Paved parking & roofs	2.06	C	98.00
Composite Area & Weighted CN	2.06		98.00

Time of Concentration

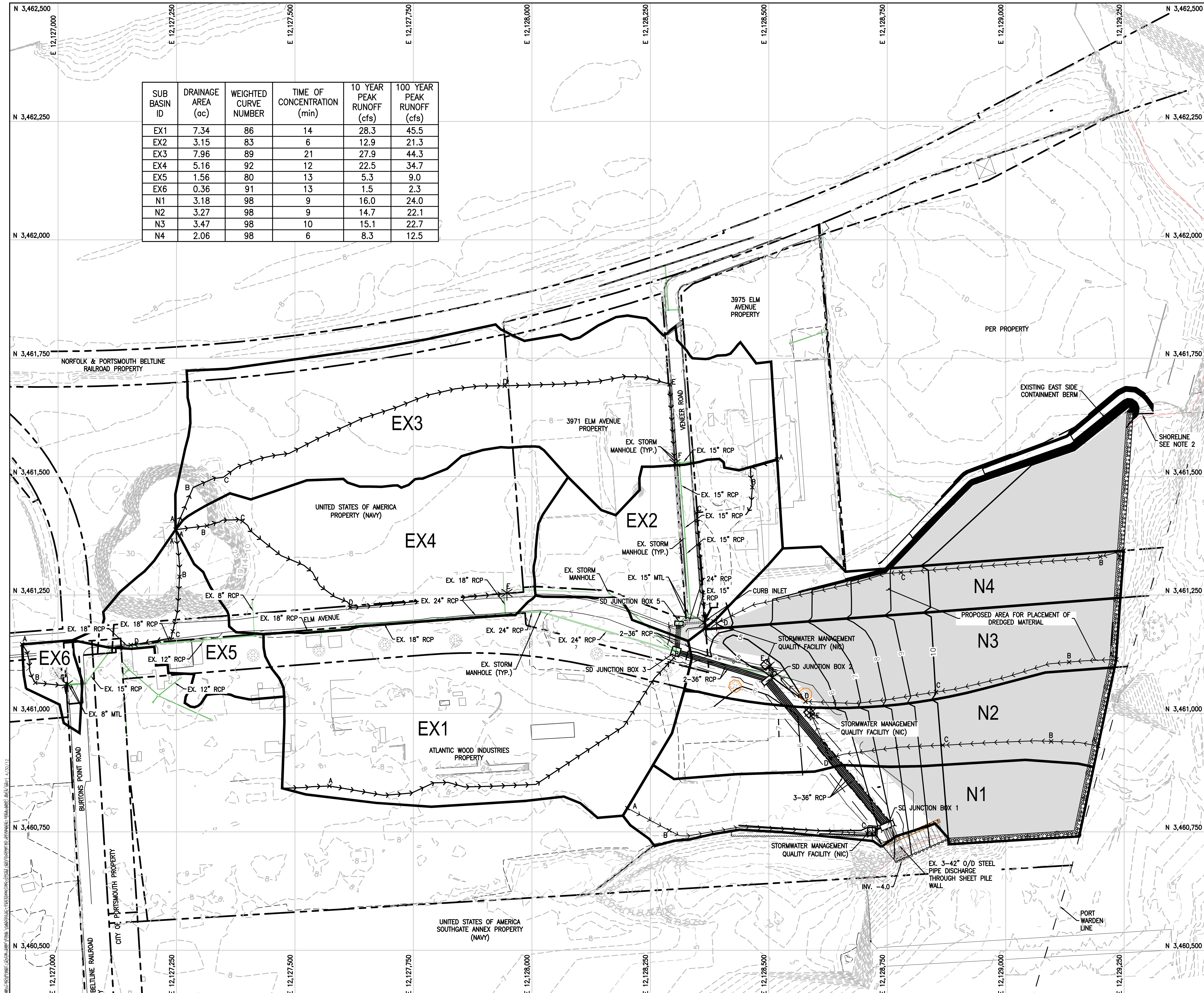
Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.01	0.00	0.00
Flow Length (ft) :	51	0.00	0.00
Slope (%) :	.1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	3.8	0.00	0.00
Velocity (ft/sec) :	0.43	0.00	0.00
Computed Flow Time (min) :	1.99	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	422	405	0.00
Slope (%) :	.1	1.23	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	0.51	1.79	0.00
Computed Flow Time (min) :	13.79	3.77	0.00

Total TOC (min) 19.55

Subbasin Runoff Results

Total Rainfall (in) 6.00
Total Runoff (in) 5.76
Peak Runoff (cfs) 8.31
Weighted Curve Number 98.00
Time of Concentration (days hh:mm:ss) 0 00:19:33



SUB BASIN ID	DRAINAGE AREA (ac)	WEIGHTED CURVE NUMBER	TIME OF CONCENTRATION (min)	10 YEAR PEAK RUNOFF (cfs)	100 YEAR PEAK RUNOFF (cfs)
EX1	7.34	86	14	28.3	45.5
EX2	3.15	83	6	12.9	21.3
EX3	7.96	89	21	27.9	44.3
EX4	5.16	92	12	22.5	34.7
EX5	1.56	80	13	5.3	9.0
EX6	0.36	91	13	1.5	2.3
N1	3.18	98	9	16.0	24.0
N2	3.27	98	9	14.7	22.1
N3	3.47	98	10	15.1	22.7
N4	2.06	98	6	8.3	12.5

LEGEND

SYMBOL — **DESCRIPTION**

- EXISTING INDEX CONTOUR
- EXISTING INTERMEDIATE CONTOUR
- EXISTING BUILDING
- EXISTING ROAD
- EXISTING GRAVEL ROAD
- EXISTING RAILROAD
- EXISTING FENCE
- EXISTING GUARD RAIL
- EXISTING FIRE HYDRANT
- EXISTING STORM DRAIN INLET
- EXISTING MONITORING WELL
- EXISTING MONITORING WELL AND BOLLARDS
- EXISTING BENCHMARK
- EXISTING CONTROL POINT
- EXISTING UTILITY POLE
- EXISTING LIGHT POLE
- EXISTING SIGN
- EXISTING BILLBOARD
- EXISTING MONUMENT
- EXISTING ELECTRIC MANHOLE
- EXISTING ELECTRIC METER
- EXISTING PULL BOX
- EXISTING TELEPHONE MANHOLE
- EXISTING TELEPHONE PEDESTAL
- EXISTING GAS LINE MARKER
- EXISTING WETLAND
- EXISTING RIPRAP
- EXISTING DITCH CENTERLINE
- EXISTING TREE OR BRUSH LINE
- SS — EXISTING SANITARY FORCEMAIN
- SAN — EXISTING SANITARY SEWER PIPE (GRAVITY)
- V — EXISTING WATER PIPE
- EXISTING STORM DRAIN
- E — EXISTING UNDERGROUND ELECTRIC LINE
- UNK — EXISTING UNDERGROUND UNKNOWN UTILITY
- O/H — EXISTING OVERHEAD ELECTRIC LINE
- UT — EXISTING UNDERGROUND TELEPHONE LINE
- OT — EXISTING OVERHEAD TELEPHONE LINE
- GAS — EXISTING GAS LINE
- STM — EXISTING STEAM LINE
- AIR — EXISTING COMPRESSED AIR LINE
- EXISTING PROPERTY LINE
- EXISTING SHORELINE
- EXISTING BRIDGE PIER
- 10 — PROPOSED INDEX CONTOUR
- 9 — PROPOSED INTERMEDIATE CONTOUR
- PROPOSED STORM DRAIN
- PROPOSED EASEMENT
- LOD — LIMIT OF DISTURBANCE
- SF — SILT FENCE
- TEMPORARY DIVERSION DIKE
- DRAINAGE AREA BOUNDARY
- TIME OF CONCENTRATION PATH
- PROPOSED DREDGED MATERIAL PLACEMENT AREA

NOTES:

1. CURRENT DEEDS FOR SHORELINE PROPERTIES OWNED BY AWI AND DEEDS FOR THE 3971/3975 ELM AVENUE PROPERTIES (AND LIKELY THE PER PROPERTY, FIGG BRIDGE DEVELOPERS, AND THE U.S. NAVY) STATE THAT THE EASTERN BOUNDARY OF EACH PROPERTY IS THE PORT WARDEN'S LINE. HOWEVER, EPA BELIEVES THAT THE SUB-TIDAL RIVER BOTTOM ADJACENT TO THESE PROPERTIES IS INCLUDED IN A 1711 GRANT FROM THE COLONIAL GOVERNMENT ON BEHALF OF QUEEN ANNE, IN WHICH THE EASTERN BOUNDARY OF THE GRANTED PROPERTY WAS DESCRIBED AS EXTENDING TO THE MIDDLE OF THE RIVER. THE VIRGINIA OFFICE OF THE ATTORNEY GENERAL (VAG) HAS INFORMED EPA THAT PROPERTY OWNERS BEAR THE BURDEN OF PROVING PRIVATE OWNERSHIP OF SUBAQUEOUS BEDS BY GRANT, AS CONTEMPLATED BY VIRGINIA CODE SECTION 28.2-1200. FURTHERMORE, THE VAG CONTENDS THAT, BECAUSE THERE IS NO VIRGINIA CASE LAW FINDING IN FAVOR OF PRIVATE OWNERSHIP OF SUBAQUEOUS BOTTOMLANDS, THE BED OF THE SOUTHERN BRANCH OF THE ELIZABETH RIVER IS COMMONWEALTH PROPERTY.

2. SHORELINE IS APPROXIMATE AND SHOULD BE USED FOR REFERENCE ONLY.

N

100 50 0 100 200

GRAPHIC SCALE IN FEET

EA ENGINEERING, SCIENCE, AND TECHNOLOGY

DESIGNED BY	DRAWN BY	DATE	PROJECT NO.
JLL	JLL	AUGUST 2012	1453011
CHECKED BY	PROJECT MGR.	DRAWING NO.	FIGURE
GAT	PAP	DAM2	2

REMEDIAL DESIGN
ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER COLLECTION TRENCH
ATLANTIC WOOD INDUSTRIES SUPERFUND SITE
PORTSMOUTH, VIRGINIA

POST DEVELOPMENT DRAINAGE MAP

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Table 1 - Post-Development Hydrologic Summary

Sub Basin ID	Drainage Area (acres)	Runoff Curve Number	Time Of Concentration (minutes)	2 year Runoff (cfs)	5 year Runoff (cfs)	10 year Runoff (cfs)	25 year Runoff (cfs)	50 year Runoff (cfs)	100 year Runoff (cfs)
EX1	7.34	86	14	16.1	24.8	28.3	34.0	39.8	45.5
EX2	3.15	83	6	7.1	11.2	12.9	15.7	18.5	21.3
EX3	7.96	89	21	16.4	24.6	27.9	33.4	38.9	44.3
EX4	5.16	92	12	13.8	20.0	22.5	26.6	30.6	34.7
EX5	1.56	80	13	2.8	4.6	5.3	6.5	7.8	9.0
EX6	0.36	91	13	0.9	1.3	1.5	1.8	2.1	2.3
N1	3.18	98	9	10.3	14.4	16.0	18.7	21.3	24.0
N2	3.27	98	9	9.5	13.2	14.7	17.2	19.7	22.1
N3	3.47	98	10	9.7	13.6	15.1	17.6	20.1	22.7
N4	2.06	98	6	5.4	7.5	8.3	9.7	11.1	12.5

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Appendix B

Dewatering Calculations

- **Dewatering Calculations**

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OBJECTIVE:

Determine the volume of water, located in the subsurface material, to be dewatered to an elevation three feet below trench bottom as part of the Elm Avenue Storm Drain Relocation project on the AWI property. The volume to be calculated represents the existing volume to be dewatered in order to begin construction of the Elm Avenue Storm Drain Relocation Project and also the maintenance flow during the construction activities. The dewatering operation calculations for the maintenance flow rate necessary to maintain a groundwater surface elevation below the trench bottom is shown in the following calculations titled "Dewatering for Storm Drain Relocation."

GIVENS AND ASSUMPTIONS:

Assumed dewatering efforts to drawdown groundwater elevation to an elevation three (3) feet below trench bottom.

Well points are positioned to be 10 feet outside liner anchor trench.

Soil characteristics based on geotechnical data found in Appendix G.

Assumed drawdown geometry for estimation purposes as shown for each cross section on Sheet 3 of 4.

Duration of the construction activities requiring dewatering operations is 5 months.

PROCEDURE:

1. **Existing Volume**
   Calculated the volume of water in the saturated subsurface material for each cross section:

Area:
Calculations for the area of each cross section are shown on Sheet 4 of 4.

Total Volume:
Total volume, including soil and water, determined from area of cross section and linear feet of trench section. Calculations shown on Sheet 4 of 4.

Water Volume:
Water volume determined using a Specific Yield of 30%. Calculations for each section shown on Sheet 4 of 4; total existing water volume is estimated as 3,200,590 gallons.

2. **Maintenance Volume**

   Calculate the volume of water required to maintain the groundwater elevation 3 feet below the trench bottom during construction activities:

Maintenance flow rate calculations for the dewatering operations during construction activities are shown in following calculations titled "Dewatering for Storm Drain Relocation."

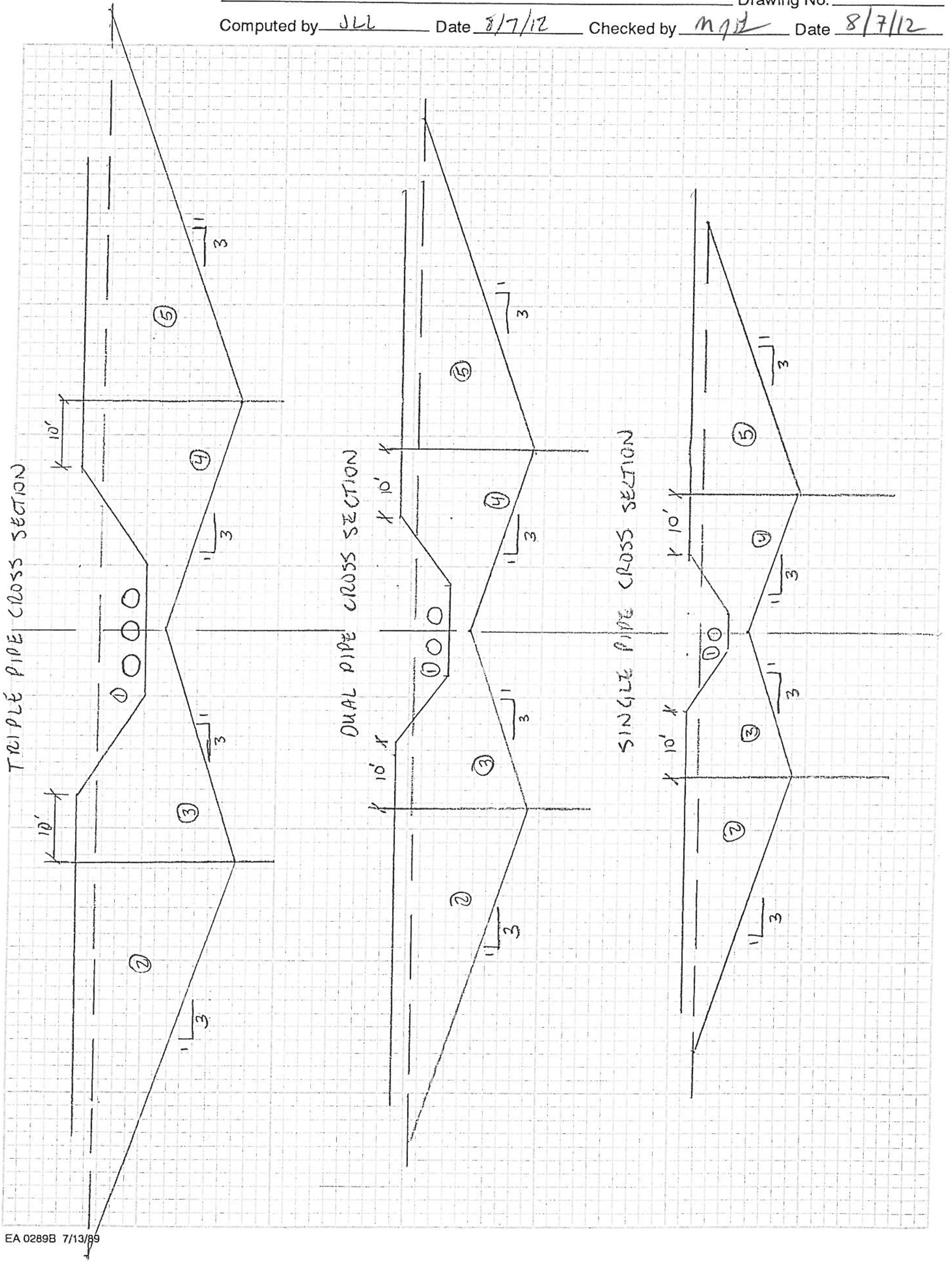
Based on the modeled flow rates for the southeast and northwest portions of the trench an average flow rate of 11 gallons per minute (gpm) is assumed for the construction of the storm drain relocation. In order to maintain a dry construction site the dewatering operations will run continuously during construction activities. Given the flow rate of 11 gpm the total volume for the construction activities is:

$$11\,\frac{gallons}{\min}\ x\ 60\,\frac{\min}{hr}\ x\ 24\,\frac{hr}{day}\ x\ 30\,\frac{day}{month}\ 5\ months = 2{,}376{,}000\ gallons$$

<u>CONCLUSIONS:</u>

Based on storm drain trench cross sections and length of trench the total volume to be dewatered as part of the initial dewatering operations (3,200,590 gallons) and the maintenance dewatering flow (2,376,000) is 5,580,000 gallons. Use 5,600,000 gallons in Bid Schedule.

Project ATLANTIC WOOD INDUSTRIES SUPERFUND SITE | Project No. 1453011
Subject DEWATERING OPERATIONS VOLUME | Sheet No. 3 of 4
Drawing No.
Computed by JLL Date 8/7/12 Checked by MJH Date 8/7/12



Atlantic Wood Industries Superfund Site
Dewatering Operations Volume

Computed by: JLL Date: 8/7/17 Checked by: ________ Date: ________

Triple Pipe Cross Section

Area ID	Shape	Depth (FT)	Length (FT)	Area (SF)
Area 1	Rectangle	10	70	700
Area 2	Triangle	20	60	600
Area 3	Triangle	11.66	35	204
Area 4	Triangle	11.66	35	204
Area 5	Triangle	20	60	600
				2308

Area (SF)	Trench Length (LF)	Total Volume (cf)	Specific Yield[1] (%)	Water Volume (cf)	Water Volume (gal)
2308	425	980943	30.0%	294283	2201382

Dual Pipe Cross Section

Area ID	Shape	Depth (FT)	Length (FT)	Area (SF)
Area 1	Rectangle	8	55	440
Area 2	Triangle	17	51	434
Area 3	Triangle	9	27	122
Area 4	Triangle	9	27	122
Area 5	Triangle	17	51	434
				1550

Area (SF)	Trench Length (LF)	Total Volume (cf)	Specific Yield[1] (%)	Water Volume (cf)	Water Volume (gal)
1550	250	387500	30.0%	116250	869608

Single Pipe Cross Section

Area ID	Shape	Depth (FT)	Length (FT)	Area (SF)
Area 1	Rectangle	7.5	42	315
Area 2	Triangle	14	42	294
Area 3	Triangle	7	21	74
Area 4	Triangle	7	21	74
Area 5	Triangle	14	42	294
				1050

Area (SF)	Trench Length (LF)	Total Volume (cf)	Specific Yield[1] (%)	Water Volume (cf)	Water Volume (gal)
1050	55	57750	30.0%	17325	129600

Total Water Volume (gal)	3200590

[1] - Specific Yield assumed based on aquifer characteristics.

Project	Atlantic Wood Industries Superfund Site	Project No.	1453011
Subject	Dewatering for Storm Drain Relocation	Sheet No.	1 of 3
		Drawing No.	

Computed by (b) (4) [redacted] Date 08/07/12 Checked by [illegible] Date 8/7/12

OBJECTIVE:

Determine the rate of extraction of water necessary to dewater the storm drain trench during drain relocation.

GIVENS AND ASSUMPTIONS:

- The trench will be dewatered in 200-ft-long sections, either to the bottom of the trench or to a depth of 3 feet below the bottom of the trench.
- Trench dimensions used in the calculation were from the March 2012 Pre-Final Design drawings.
- The groundwater model developed for the AWI site was used for portions of the calculations. For details on the parameters and assumptions of the model, see Appendix A of the Groundwater Management Alternative Analysis.

PROCEDURE:

Calculations were performed for two areas along the path of the proposed trench: one 200 ft section in the southeastern portion of the trench, near the outlet to the river, and the other in the northwestern portion of the trench.

The AWI groundwater model (calibrated to historical conditions) was used to obtain a numerical solution for the pumping rate necessary to maintain the desired groundwater depression. Drains at the elevation of the bottom of the trench were used in the model to simulate pumping of the aquifer.

Southeastern Section

Used the AWI groundwater model to calculate the flow rate (cumulative) into drains spaced 25 feet apart over a 200-foot-long, 50-ft-wide southeastern section of the trench (bounded by the red lines in Figure 1 below), with a bottom elevation of either -4 feet msl (bottom of trench) or -7 feet msl (3 feet below bottom of trench).

Modeled flow rate needed to maintain a dewatered condition at elevation -4 feet msl: 1,784 cubic feet/day = 9.3 gallons/minute

Modeled flow rate needed to maintain a dewatered condition to elevation -7 feet msl: 2,396 cubic feet/day = 12.4 gallons/minute

Project	Atlantic Wood Industries Superfund Site	Project No.	1453011
Subject	Dewatering for Storm Drain Relocation	Sheet No.	2 of 3
		Drawing No.	
Computed by	(b) (4) [redacted] Date 08/07/12	Checked by	m7L Date 8/7/12

Northwestern Section

Used the AWI groundwater model to calculate the flow rate (cumulative) into drains spaced 25 feet apart over a 200-foot-long, approximately 30-ft-wide northwestern section of trench (bounded by red lines in Figure 2 below), with a bottom elevation of either -2.5 feet msl (bottom of trench) or -5.5 feet msl (3 feet below bottom of trench).

Modeled flow rate needed to maintain a dewatered condition at elevation -2.5 feet msl: 1,200 cubic feet/day = 6.2 gallons/minute

Modeled flow rate needed to maintain a dewatered condition at elevation -5.5 feet msl: 1,810 cubic feet/day = 9.4 gallons/minute

CONCLUSIONS:

The rates of water extraction necessary per 200-ft-long trench section are approximately 6-9 gallons/minute to maintain a groundwater elevation at the bottom of the trench, or approximately 9-12 gallons/minute to maintain a groundwater elevation 3 feet below the bottom of the trench. The necessary rate of extraction is higher in the southeastern portion of the trench than in the northwestern portion.

Project	Atlantic Wood Industries Superfund Site	Project No.	1453011
Subject	Dewatering for Storm Drain Relocation	Sheet No.	3 of 3
		Drawing No.	
Computed by (b) (4) [redacted] Date 08/07/12		Checked by MJL	Date 8/7/12

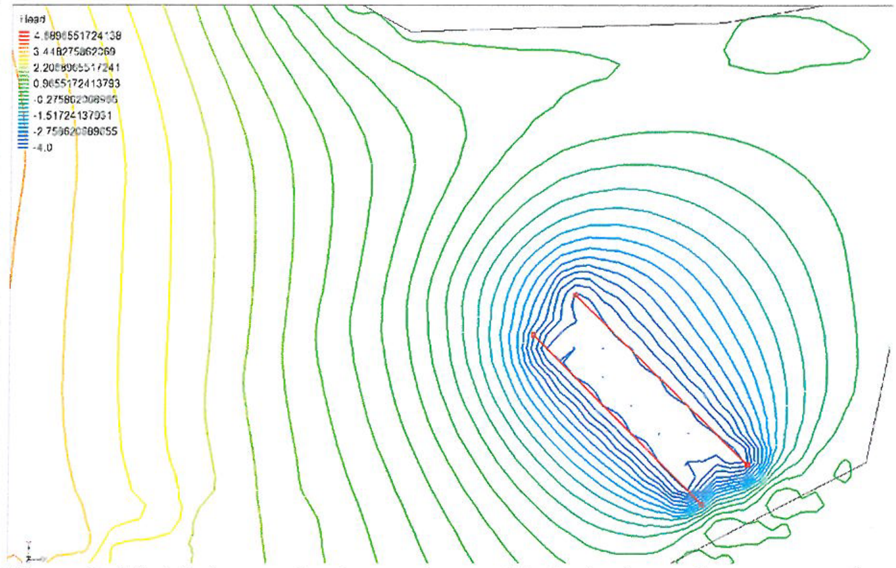


Figure 1: Modeled groundwater contours with drains in southeastern portion.

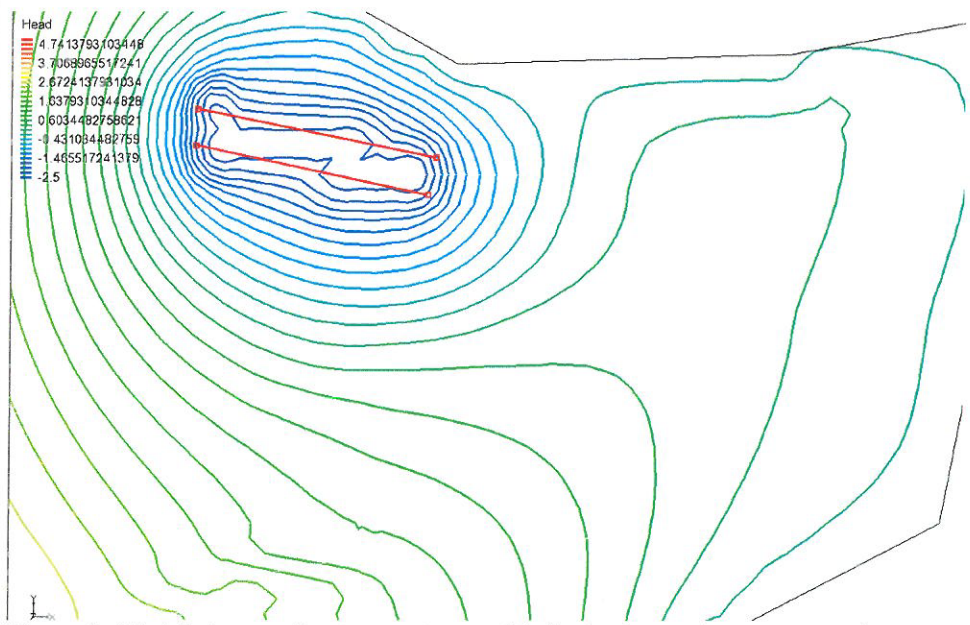


Figure 2: Modeled groundwater contours with drains in northwestern portion.

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Appendix C

Geomembrane Liner Calculations

- **HDPE Chemical Compatibility**
- **Liner Slope Stability Calculations**
- **Anchor Trench Calculations**
- **Geomembrane Puncture Calculations**

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Geomembrane Liner Calculations
HDPE Chemical Compatibility

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The information provided varies from table to table since different information is relevant and/or available for each material. The following grid outlines all of the different fields which can be found in the data tables.

FIELD HEADING	DESCRIPTION
Exposure medium	reagent or other medium to which the thermoplastic was exposed
Exp. medium note	additional Information about the exposure medium and conditions of exposure
Conc. (%)	concentration of the given exposure medium, often expressed as a percentage
Temp. (°C)	exposure temperature in degrees °C
Time (days)	exposure time in days
PDL #	PDL Rating; based on a scale of 0 to 9 (with 9 as the highest resistance); details of how the rating is calculated are given later in this section
% Change	
Length	% change from length before exposure
Vol.	% change from volume before exposure
Weight	% change from weight before exposure
% Retained	
Elong.	% of original elongation retained
Tensile strength	% of original tensile strength retained
Impact strength	% of original impact strength retained
Resistance note	additional Information about the resistance of the thermoplastic to the exposure medium (i.e. observed changes, safety notes, etc.)
Test note	additional information about the test
Material note	details of the specific material tested, this includes, if available, supplier, trade name, grade, filler, specimen details (although supplier names in particular may have changed since the data was compiled).

Weighted Value	Weight Change*	Diameter; Length* Change	Volume Change*	Mechanical** Property Retained	Visual/Observed*** Change
10	0–0.25	0–0.1	0–2.5	>=97	no change
9	>0.25–0.5	>0.1–0.2	>2.5–5.0	94–<97	
8	>0.5–0.75	>0.2–0.3	>5.0–10.0	90–<94	
7	>0.75–1.0	>0.3–0.4	>10.0–20.0	85–<90	slightly discolored slightly bleached
6	>1.0–1.5	>0.4–0.5	>20.0–30.0	80–<85	discolored yellows slightly flexible
5	>1.5–2.0	>0.5–0.75	>30.0–40.0	75–<80	possible stress crack agent flexible possible oxidizing agent slightly crazed
4	>2.0–3.0	>0.75–1.0	>40.0–50.0	70–<75	dostorted, warped softemed slight swelling blistered known stress crack agent
3	>3.0–4.0	>1.0–1.5	>50.0–70.0	60–<70	cracking, crazing brittle plasticizer oxidizer softened swelling surface hardened
2	>4.0–6.0	>1.5–2.0	>60.9–90.0	50–<60	severe distortion oxidizer and plasticizer deteriorated
1	>6.0	>2.0	>90.0	>0–<50	decomposed
				0	solvent dissolved disintegrated

*All values are given as percentage change from original.

**Percentage mechanical properties retained include tensile strength, elongation, modulus, flexural strength and impact strength. If the % retention is greater than 100%, a value of 200 minus the % property retained is used in the calculations.

***Due to the variety of information of this type reported, this information can be used only as a guideline.

Exposure medium	Exp. medium note	Conc. (%)	Temp. (°C)	Time (days)	PDL #	Resistance note	% Change		% Retained		Test note	Material note
							Vol.	Weight	Elong.	Tensile strength		
Coumarone Resins (cont)			60	60	8	Resistant	<3	<0.5	>85			Hostalen HMW; Hoechst Celanese; Specimen: 50×25×1 mm (2×1×0.04 in) from press-molded sheets to DIN 53455
			60		9	Resistant; tensile strength at yield and elongation at break unchanged					Test specimen 1B according to ISO 527-2	Lupolen; Basell; Specimen: 50×25×1 mm
Cranberry Sauce			21		8	Satisfactory resistance						Fortiflex; Solvay
			60		8	"						"
Cream	face; hands		20		9	Good resistance; the product has no effect						SABIC HDPE; SABIC
	"		60		9	"						"
Creolin			20		9	"						"
			60		9	"						"
Creosote			20	60	8	Resistant	<3	<0.5				Hostalen; Hoechst Celanese; Specimen: 50×25×1 mm (2×1×0.04 in) from press-molded sheets to DIN 53455
			20	60	8	"	<3	<0.5	>85			Hostalen HMW; Hoechst Celanese; Specimen: 50×25×1 mm (2×1×0.04 in) from press-molded sheets to DIN 53455
			20		9	Resistant; tensile strength at yield and elongation at break unchanged					Test specimen 1B according to ISO 527-2	Lupolen; Basell; Specimen: 50×25×1 mm
			20		9	Good resistance; the product has no effect						SABIC HDPE; SABIC
			60	60	8	Resistant/possible discoloration	<3	<0.5				Hostalen; Hoechst Celanese; Specimen: 50×25×1 mm (2×1×0.04 in) from press-molded sheets to DIN 53455

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Geomembrane Liner Calculations
Liner Slope Stability Calculations

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Project	AWI Remedial Design	Project No.	14530.11
Subject	Liner Slope Stability	Sheet No.	1 of 3
		Drawing No.	
Computed by TJP	Date 6/19/12	Checked by SMD	Date 6/19/12

OBJECTIVE:

Determine the stability of the liner system for the storm drain trench. Design the anchor trench to hold the liner system in place and verify the tension in the liner system does not exceed its strength.

PROCEDURE:

See attached calculations

1) Calculate the pullout force that would result in anchor trench failure based on the trench geometry and soil and geosynthetic properties. Utilize interface friction angles from Geosynthetic Research Institute Report #30, "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces," June 14, 2005.

See attached calculations

2) Calculate the potential tension in the liner system under the condition whereby fill has yet to be installed in the trench. The tension is based on the weight of the geosynthetic materials.

3) Verify that the tension in the liner system does not exceed the pullout force for the anchor trench. Calculate the factor of safety.

Anchor trench pullout force = 70 lb/in
Liner system tension = 0.2 lb/in
Factor of safety against liner pullout = 70 / 19 = 350

4) Verify that the tension in the liner system components does not exceed the strength of the liner system materials.

Geotextile strength at yield = 330 lb/in — Minimum allowed in specs
Liner strength at yield = 126 lb/in — Minimum allowed in specs
Liner system tension = 0.2 lb/in
Factor of safety against geotextile yielding = 330 / 0.2 = 1,650
Factor of safety against liner yielding = 126 / 0.2= 630

CONCLUSION:

The trench liner system will be stable before fill is placed in the trench.

AWI Remedial Design - 2 of 3
TENSION IN LINER IN STORM DRAIN TRENCH

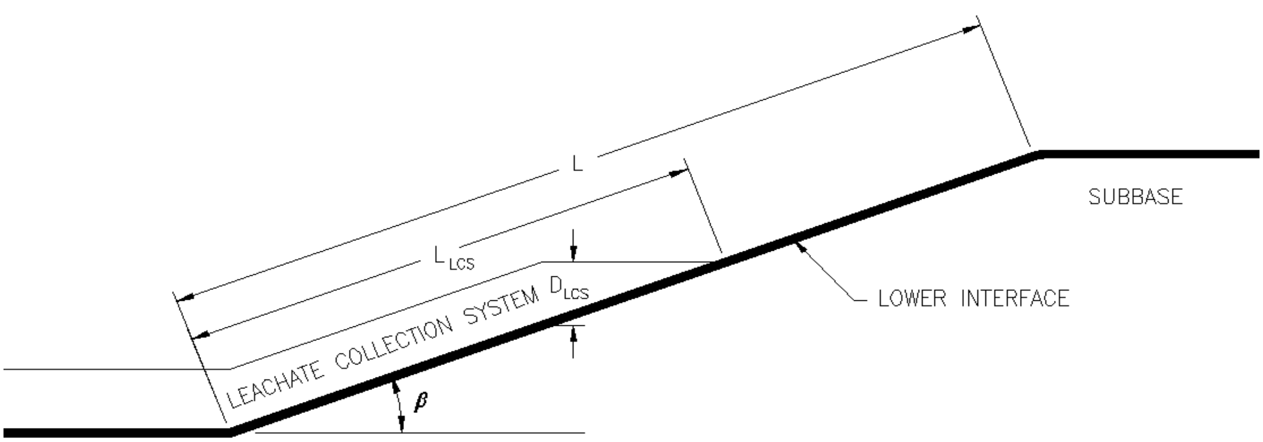
slope length, L =	15.9	ft	4.846	m
GCL weight =	0	psf		
liner density, γ_L =	0.94	g/cm^3		
liner thickness, t =	60	mil		
geotextile weight =	32	oz/yd^2	(16 oz. geotextiles, 2 each)	
Trench Bedding unit weight, γ_{LCS} =	120	pcf	18.85	kN/m^3
depth of LCS, D_{LCS} =	2.25	ft	0.686	m
length of LCS, L_{LCS} =	0	ft	0	m
lower interface friction angle, δ =	21	°		Nonwoven geotextile / subbase
slope, z =	1.5	H:1V		
slope, β =	33.69	°		

$$T_L = W \times (\sin\beta - \cos\beta \tan\delta)$$

where,
W = weight of liner system and leachate collection system on side slope

LINER TENSION

T_L =	**2**	**lb/ft**
	0.2	**lb/in**
	0.0	**kN/m**



AWI Remedial Design - 3 of 3
ANCHOR TRENCH PULLOUT FORCE

unit weight of soil, γ =	120	pcf	18.85	kN/m^3
depth of soil, D =	0	ft		Conservative assumption
depth of anchor trench, D_A =	2	ft	0.61	m
internal friction angle, ϕ =	26	°		Anchor trench soil
lower interface friction angle, δ =	21	°		Nonwoven geotextile / subbase
slope, z =	1.5	H:1V		
slope, β =	33.69	°		
length of runout, L =	1	ft	0.305	m
normal stress on liner runout, σ_n =	0	psf	0	kN/m^2
passive pressure in trench, P_P =	614.7	lb/ft	8.97	kN/m
active pressure in trench, P_A =	93.71	lb/ft	1.368	kN/m

$$\boxed{F_{PO} = \frac{\sigma_n \tan\delta \times L + P_P - P_A}{\cos\beta - \sin\beta \tan\delta}}$$

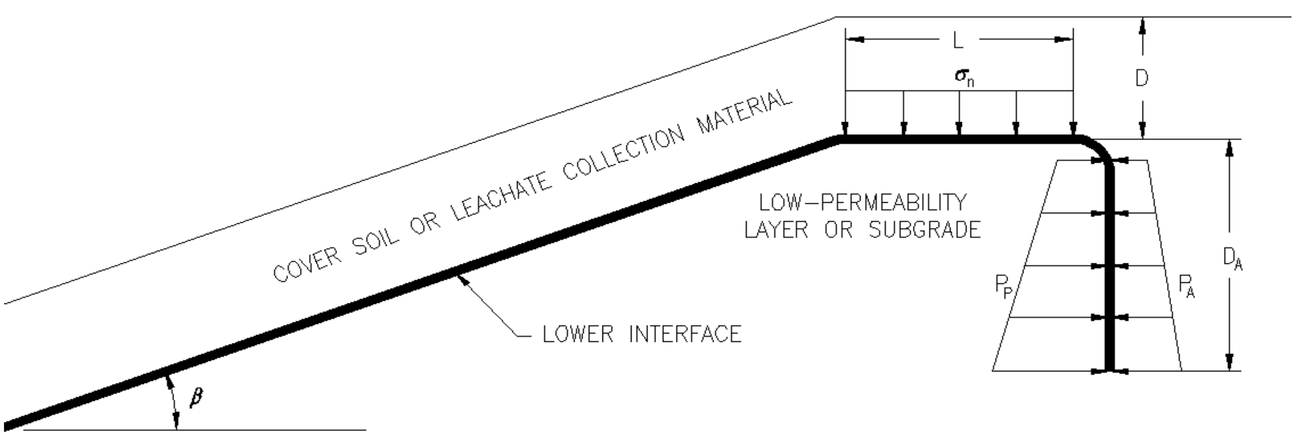
PULLOUT FORCE

F_{PO} = 841 lb/ft

70 lb/in

12.3 kN/m

Reference: Designing with Geosynthetics, 4th edition, Koerner, Robert M., 1998.



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Geomembrane Liner Calculations
Anchor Trench Calculations

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Project: AWI Remedial Design, Elm Avenue Storm Drain Relocation and Groundwater Collection Trench
Subject: Anchor Trench
Computed by: <u>TJP</u> *Checked by:* <u>SMD</u>

Unit conversions

12	in	=	304.8	mm	(Thickness of cover soil)
1	ft	=	0.3048	m	(Embedment length)
2	ft	=	0.6096	m	(Depth of anchor trench)
115	pcf	=	18.0665	kN/m^3	(Soil unit weight)
60	mil	=	1.524	mm	(Thickness of geomembrane)
2200	psi	=	15168.34	kPa	(Allowable stress in geomembrane)

Assumptions

1. Friction angle of subgrade is 22 degrees (high plasticity soils)
2. Friction angle of low permeability gravel is 40 degrees (well graded gravel)
3. Friction angle of compacted material installed within anchor trench is 35 degrees
4. Allowable stress in HDPE geomembrane is 2200 psi per Inherent Properties of Polyethylene Liners listed under specifications by Poly-Flex, Inc.

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Design Calculator

Anchor Trench

Problem Statement

Anchorage is designed to prevent wind and water from moving under the geomembrane; it is not designed to allow geomembranes to be tensioned. The anchor trench design should allow pullout of the geomembrane before tension failure. This is directly reflected in the anchorage ratio:

$$AR = \frac{T_{GMallow}}{T_{ATallow}}$$

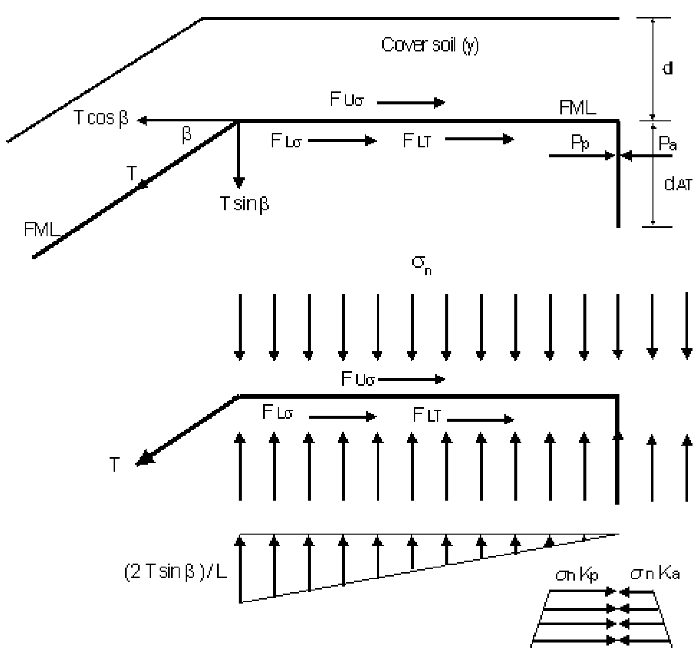
AR	Anchorage Ratio
$T_{GM\ allow}$	Allowable geomembrane tension from ASTM D3886
$T_{AT\ allow}$	Allowable concrete anchor trench tension from analytic model

Anchorage Ratio > 1	Geomembrane pull-out mode controls
Anchorage Ratio = 1	Balanced Design
Anchorage Ratio < 1	Geomembrane tension rupture mode controls

$$T_{GMallow} = \sigma_{allow} * t$$

$$\sigma_{allow} = \frac{\sigma_{ult}}{FS}$$

$$T_{ATallow} = \frac{[(\tan\delta_U + \tan\delta_L)*(\gamma * d)]*L + (K_P - K_A)*[0.5*\gamma*d_{AT}^{\ 2} + d*\gamma*d_{AT}]}{\cos\beta - \sin\beta * \tan\delta_L}$$



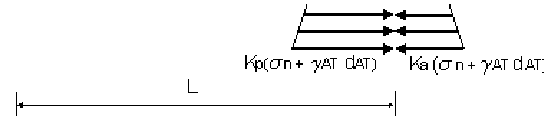


Figure 1- **Cross section of anchor trench section and related stresses and forces involved**

Note that the factor of safety is placed on the geomembrane force T, which is used as an allowable value.

σ_{allow}	The allowable geomembrane stress
t	The geomembrane thickness
σ_{ult}	The ultimate geomembrane stress, e.g., yield or break
$T_{ATallow}$	Allowable anchor trench tension
γ_{AT}	Soil unit weight
d_{AT}	Depth of the anchor trench
d	Thickness of the cover soil
L	Embedment length
δ_L	FML / soil friction angle (below geomembrane)
δ_U	Cover soil / geomembrane friction angle (above geomembrane)
Φ	Soil internal friction angle
β	Side slope angle
K_p	Coefficient of passive earth pressure = $\tan^2(45^o+\Phi/2)$
K_a	Coefficient of active earth pressure = $\tan^2(45^o-\Phi/2)$
FS	The factor of safety for geomembrane against tension response
$F_{U\sigma}$	Shear force above geomembrane due to cover soil (for thin cover soils tensile cracking will occur and this value will then be negligible)
$F_{L\sigma}$	Shear force below the geomembrane due to cover soil
F_{LT}	Shear force below geomembrane due to vertical component of T_{allow}
σ_n	Applied normal stress from the cover soil

Input Values

Geometry

Side slope angle (β)	33.69	**degrees**
Thickness of the cover soil (d)	304.8	**mm**
Embedment length (L)	.3048	**m**
Depth of the anchor trench (d_{AT})	.6096	**mm**
Soil Properties		
FML / soil friction angle (δ_L)	22	**degrees**
Cover soil / FML friction angle (δ_U)	40	**degrees**
Soil friction angle (Φ)	35	**degrees**
Soil unit weight (γ)	18.81	**kN/m^3**
Geomembrane Properties		
Thickness geomembrane (t)	1.524	**mm**
Allowable stress in geomembrane (σ_{allow})	15168	**kPa**

Calculate

Solution

Allowable anchor tension	**18.86 kN/m**
Allowable geomembrane tension	**23.12 kN/m**
Anchorage ratio	**1.23**

Assistance

References

"Designing with Geosynthetics". **R.M. Koerner**, Prentice Hall Publishing Co., Englewood Cliffs, NJ, 1998.

"Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments", **G. N. Richardson and R. M. Koerner**, 1987.

Geomembrane Liner Calculations
Geomembrane Puncture Calculations

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Project: AWI Remedial Design, Elm Avenue Storm Drain Relocation and Groundwater Collection Trench
Subject: Safety Factor Against Geomembrane Puncture
Computed by: TJP *Checked by:* SMD

Unit conversions

16	oz	=	453.59	g	
1	sy	=	0.83613	m^2	
16	oz/sy	=	542.4874	g/m^2	(Geotextile mass per unit area)
11.6	ft	=	3.53568	m	(Depth of material on top of geomembrane)
115	pcf	=	18.0665	kN/m^3	(Unit weight of select fill material within Triple Pipe Trench)
182.6816	pcf	=	28.69927	kN/m^3	(Unit weight of area within Triple Pipe Trench occupied by RCP)
119.7377	pcf	=	18.81079	kN/m^3	(Adjusted unit weight to account for weight of RCP)

Assumptions

1. Modification factor for protrusion shape is 0.5 (subround)
2. Modification factor for packing density is 0.83 (Dense)
3. Modification factor for arching in solids is 1.0 (Hydrostatic)
4. Modification factor for long-term creep is 1.5
5. Modification factor for chemical/biological degradation is 1.1 (Mild leachate)

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Design Calculator

Safety Factor Against Geomembrane Puncture

Problem Statement

There are many circumstances where geomembranes are placed on or beneath soils containing relatively large-sized stones. For example, poorly prepared soil subgrade with stones protruding from the surface, and cases where crushed-stoned drainage layers are to be placed above the geomembrane.

In all of these situations, a nonwoven needle-punched geotextile can provide significant puncture protection to the geomembrane. The issue of determining the required mass per unit area of the geotextile becomes critical.

The method presented herein (Koerner, 1998) focuses on the protection of 1.5 mm thick HDPE geomembranes. The method uses the design by function approach.

$$FS = \frac{p_{allow}}{p_{act}}$$

FS	factor of safety against geomembrane puncture
P_{act}	actual pressure due to the landfill contents or surface impoundment
P_{allow}	allowable pressure using different types of geotextiles and site specific conditions.

p_{allow} is determined by the following equation:

$$P_{allow} = \left(50 + 0.00045\frac{M}{H^2}\right)\left[\frac{1}{MF_S * MF_{PD} * MF_A}\right]\left[\frac{1}{RF_{CR} * RF_{CBD}}\right]$$

Symbol	Name	Unit
P_{allow}	allowable pressure	kPa
M	geotextile mass per unit area	g/m2
H	height of the protrusion above the subgrade	m
MF_S	modification factor for protrusion shape	-
MF_{PD}	modification factor for packing density	-
MF_A	modification factor for arching in solids	-
RF_{CR}	reduction factor for long-term creep	-
RF_{CBD}	reduction factor for long-term chemical/biological degradation	-

p_{allow} is determined by Modification Factors and Reduction Factors for Geomembrane Protection Design Using Nonwoven Needle-Punched Geotextile

MF_S		MF_{PD}		MF_A	
Angular:	1.0	Isolated	1.0	Hydrostatic	1.0
Subrounded:	0.5	Dense, 38 mm	0.83	Geostatic, shallow	0.75
Rounded:	0.25	Dense, 25 mm	0.67	Geostatic, mod.	0.50
		Dense, 12mm	0.50	Geostatic, deep	0.25

RF_{CBD}		RF_{CR}			
		Protrusion (mm) / Mass per Unit area (g/m^2)	38	25	12
Mild leachate	1.1	Geomembrane alone	N/R	N/R	N/R
Moderate leachate	1.3	270	N/R	N/R	>1.5
Harsh leachate	1.5	550	N/R	1.5	1.3
		1100	1.3	1.2	1.1
		>1100	1.2	1.1	1.0

N/R = Not Recommended

Input Values

M	542.49	**Geotextile mass per unit area (g/m2)**
d	3.53	**depth of material on top of geomembrane (m)**
γ	18.81	**Unit weight of material on top of geomembrane (kN/m3)**
H	0.025	**Protrusion height (m)**

Modification and Reduction Factors

MF_S	0.5
MF_{PD}	.83
MF_A	1.0
RF_{CR}	1.5
RF_{CBD}	1.1

Calculate

Solution

Factor of Safety against Geomembrane Puncture: **9.69**

Assistance

References

Wilson-Fahmy, R.F., Narejo, D. and Koerner, R.M. (1996), "Puncture Protection of Geomembranes Part I: Theory", *Geosynthetics International*, Vol. 3, No. 5, pp. 605-628.

Narejo, D. and Koerner, R.M. and Wilson-Fahmy, R.F., (1996), "Puncture Protection of Geomembranes Part II: Experimental", *Geosynthetics International*, Vol. 3, No. 5, pp. 629-653.

Koerner, R.M., Wilson-Fahmy, R.F. and Narejo, D. (1996), "Puncture Protection of Geomembranes Part III: Examples", *Geosynthetics International*, Vol. 3, No. 5 pp. 655-675.

Koerner, R.M. (1998), *Designing with Geosynthetics*, Prentice Hall Publishing Co., Englewood Cliffs, NJ.

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Appendix D

Storm Drain Calculations

- **Junction Box Calculations**
- **Storm Drain Loading Calculations**
- **Storm Drain Buoyancy Calculations**

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Storm Drain Calculations
Junction Box Calculations

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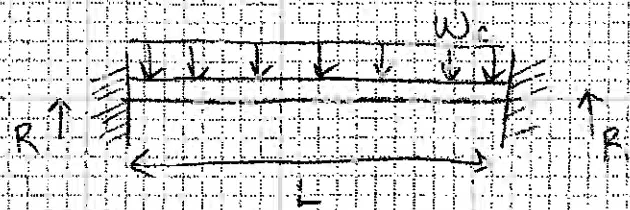
Project AWI Project No.
Subject Roof Moment Calculation Sheet No. 1 of 22
Stormwater Drain Junction Boxes Drawing No.
Computed by TCC Date 2-29-12 Checked by MT/HH Date 5-31-12

Roof Condition 1



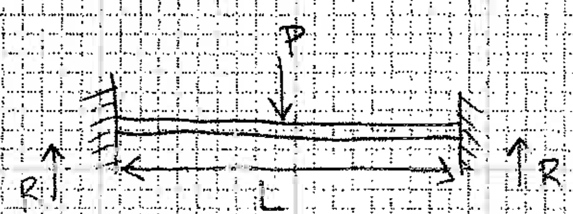
W = uniform load for Crane or H-20 truck

$$R = \frac{WL}{2}$$

$\boxed{\text{Eqn 1}}$ $M_{max\ at\ ends} = \frac{WL^2}{12}$ (top/outside corner steel)

$\boxed{\text{Eqn 2}}$ $M_{center} = \frac{WL^2}{24}$ (bottom middle)

Roof Condition 2

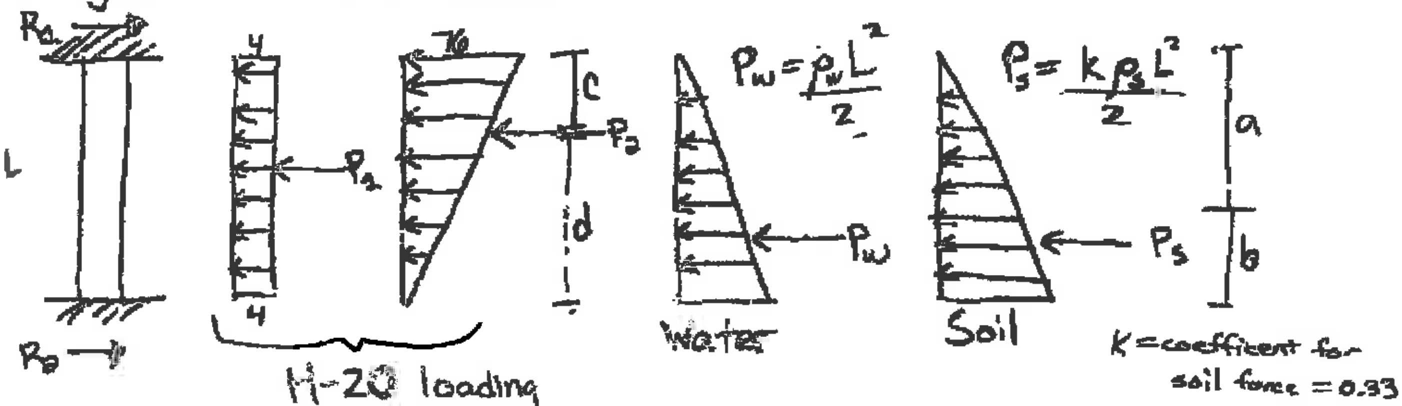


P = largest individual crane wheel load or H-20 Truck

$\boxed{\text{Eqn 3}}$ $M_{max\ at\ center/ends} = \frac{PL}{8}$

Project AWI Project No.
Subject Wall Moment Calculation Sheet No. 2 of 22
STORMWATER DRAIN JUNCTION BOXES Drawing No.
Computed by MET Date 3-2-12 Checked by HH Date 3-5-12
TCC 5/31/12

Diagram - Soil Pressure



$b = \frac{1}{3}L$ $a = \frac{2}{3}L$ $c = b$

$c = \frac{1}{3}L$ $d = \frac{2}{3}L$ $x = d$

$$R_1 = \frac{P_1}{2} + \frac{P_2 d^2}{L^3}(d+3c) + \frac{P_w b^2}{L^3}(3a+b) + \frac{P_s b^2}{L^3}(3a+b) \qquad \boxed{\text{Eqn. 4}}$$

$$R_2 = \frac{P_1}{2} + \frac{P_2 c^2}{L^3}(3d+c) + \frac{P_w a^2}{L^3}(a+3b) + \frac{P_s x^2}{L^3}(a+3b) \qquad \boxed{\text{Eqn. 5}}$$

$$M_1 = \frac{P_1 L}{12} + \frac{P_2 d^2 c}{L^2} + \frac{P_w a b^2}{L^2} + \frac{P_s a b^2}{L^2} \qquad \boxed{\text{Eqn. 6}}$$

$$M_2 = \frac{P_1 L}{12} + \frac{P_2 d c^2}{L^2} + \frac{P_w a^2 b}{L^2} + \frac{P_s a^2 b}{L^2} \qquad \boxed{\text{Eqn. 7}}$$

H-20 = 32,000 lb/axle (AASHTO 3.30) SEE PAGE 3
or 16,000 lb/wheel point load

W_s = surface pressure = 16,000 lb/200 in^2 (200 in^2 is AASHTO 3.30 the contact area)
= 80 psi

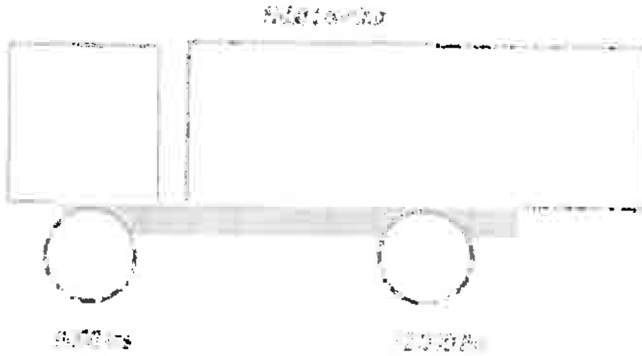
P_1 assumes 5% of H-20 load reaches the bottom of the wall
P_2 distributes remaining H-20 load along height of wall

AWI

STORMWATER DRAIN JUNCTION BOXES

HH 5/30/12 TCL 5/31/12

H-20 Loading



P = max wheel load
= 32,000 lbs / 2 tires
P = 16,000 lbs

W_c = surface pressure = P / A
Use AASHTO 3.30 Tire
Contact Area of 200 in^2
= 16,000 lbs / 200 in^2
W_c = 80 psi

ALL JUNCTION BOXES DESIGNED FOR H-20 LOADING

Project AWI
Subject Rebar Sizing Calculation Sheet No. 4 of 12
Computed by TCC Date 8-29-12 Checked by MT / HH Date 5/31/12

REBAR SIZING:

USE 4,000 psi concrete.
60,000 psi steel

$K_u = \frac{M_u}{F}$ $\boxed{Eq. 8}$ M_u = applied design moment

F = coefficient Flexure 5 pg. 115
use b = 12" d = thickness (min) value 4"

calculate $K_u = \frac{M_u}{F}$

use Flexure 12 to determine ρ
pg. 106

$\rho_{min} = \frac{200}{f_y} = \frac{200}{60{,}000} = 0.0033$

CALCULATE $A_s = \left(\underset{\text{use larger}}{\rho \text{ or } \rho_{min}}\right) b\, d$ $\boxed{Eq. 9}$

SELECT REBAR SIZE Reinforcement 14 pg. 162

AWI

STORMWATER DRAIN JUNCTION BOXES
TCC 2-29-12 MT 5/31/12

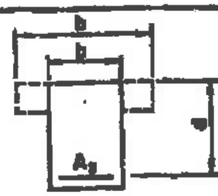
FLEXURE 5

Flexure 5—Coefficient F for resisting moments of rectangular and T-sections

Values of $F = \frac{bd^2}{12,000}$

a. Enter table with known values of $F = \frac{M_u}{K_u}$ - Select b and d in.)

b. Enter table with known value of b and d, compute resisting moment in concrete: $K_u \times F$ (ft-kips)



44

4' 6"

d ≥ 42

d	b: Width of compressive area																				
	4.0	5.0	6.0	7.0	7.5	8.0	9.0	9.5	10.0	11.0	12.0	13.0	15.0	17.0	19.0	21.0	23.0	25.0	30.0	36.0	42.0	48.0
5.0	.008	.010	.013	.015	.016	.017	.019	.020	.021	.023	.025	.027	.031	.035	.040	.044	.048	.052	.063	.075	.088	.100
5.5	.010	.013	.015	.018	.019	.020	.023	.024	.025	.028	.030	.033	.038	.043	.048	.053	.058	.063	.076	.091	.106	.121
6.0	.012	.015	.018	.021	.023	.024	.027	.029	.030	.033	.036	.039	.045	.051	.057	.063	.069	.075	.090	.108	.126	.144
6.5	.014	.018	.021	.025	.026	.028	.032	.033	.035	.039	.042	.046	.053	.060	.067	.074	.081	.088	.106	.127	.148	.169
7.0	.016	.020	.025	.029	.031	.033	.037	.039	.041	.045	.049	.053	.061	.069	.078	.086	.094	.102	.123	.147	.172	.196
7.5	.019	.023	.028	.033	.035	.038	.042	.045	.047	.052	.056	.061	.070	.080	.089	.098	.108	.117	.141	.169	.197	.225
8.0	.021	.027	.032	.037	.040	.043	.048	.051	.053	.059	.064	.069	.080	.091	.101	.112	.123	.133	.160	.192	.224	.256
8.5	.024	.030	.036	.042	.045	.048	.054	.057	.060	.066	.072	.078	.090	.102	.114	.126	.138	.151	.181	.217	.253	.289
9.0	.027	.034	.041	.047	.051	.054	.061	.064	.068	.074	.081	.088	.101	.115	.128	.142	.155	.169	.203	.243	.284	.324
9.5	.030	.038	.045	.053	.056	.060	.068	.071	.075	.083	.090	.098	.113	.128	.143	.158	.173	.188	.226	.271	.316	.361
10.0	.033	.042	.050	.058	.063	.067	.075	.079	.083	.092	.100	.108	.125	.142	.158	.175	.192	.208	.250	.300	.350	.400
10.5	.037	.046	.055	.064	.069	.074	.083	.087	.092	.101	.110	.119	.138	.156	.175	.193	.211	.230	.276	.331	.386	.441
11.0	.040	.050	.061	.071	.076	.081	.091	.096	.101	.111	.121	.131	.151	.171	.192	.212	.232	.252	.303	.363	.424	.484
11.5	.044	.055	.066	.077	.083	.088	.099	.105	.110	.121	.132	.143	.165	.187	.209	.231	.253	.276	.331	.397	.463	.529
12.0	.048	.060	.072	.084	.090	.096	.108	.114	.120	.132	.144	.156	.180	.204	.228	.252	.276	.300	.360	.432	.504	.576
12.5	.052	.065	.078	.091	.098	.104	.117	.124	.130	.143	.156	.169	.195	.221	.247	.273	.299	.326	.391	.469	.547	.625
13.0	.056	.070	.085	.099	.106	.113	.127	.134	.141	.155	.169	.183	.211	.239	.268	.296	.324	.352	.423	.507	.592	.676
13.5	.061	.076	.091	.106	.114	.122	.137	.144	.152	.167	.182	.197	.228	.258	.289	.319	.349	.380	.456	.547	.638	.729
14.0	.065	.082	.098	.114	.123	.131	.147	.155	.163	.180	.196	.212	.245	.278	.310	.343	.376	.408	.490	.588	.686	.784
14.5	.070	.088	.105	.123	.131	.140	.158	.166	.175	.193	.210	.228	.263	.298	.333	.368	.403	.438	.526	.631	.736	.841
15.0	.075	.094	.113	.131	.141	.150	.169	.178	.188	.206	.225	.244	.281	.319	.356	.394	.431	.469	.563	.675	.788	.900
15.5	.080	.100	.120	.140	.150	.160	.180	.190	.200	.220	.240	.260	.300	.340	.380	.420	.460	.501	.601	.721	.841	.961
16.0	.085	.107	.128	.149	.160	.171	.192	.203	.213	.235	.256	.277	.320	.363	.405	.448	.491	.533	.640	.768	.896	1.02
16.5	.091	.113	.136	.159	.170	.182	.204	.216	.227	.250	.272	.295	.340	.386	.431	.476	.522	.567	.681	.817	.953	1.09
17.0		.120	.145	.169	.181	.193	.217	.229	.241	.265	.289	.313	.361	.409	.458	.506	.554	.602	.723	.867	1.01	1.16
17.5		.128	.153	.179	.191	.204	.230	.242	.255	.281	.306	.332	.383	.434	.485	.536	.587	.638	.766	.919	1.07	1.23
18.0		.135	.162	.189	.203	.216	.243	.257	.270	.297	.324	.351	.405	.459	.513	.567	.621	.675	.810	.972	1.13	1.30
18.5		.143	.171	.200	.214	.228	.257	.271	.285	.314	.342	.371	.428	.485	.542	.599	.656	.713	.856	1.03	1.20	1.37
19.0			.181	.211	.226	.241	.271	.286	.301	.331	.361	.391	.451	.511	.572	.632	.692	.752	.903	1.08	1.26	1.44
20.0			.200	.233	.250	.267	.300	.317	.333	.367	.400	.433	.500	.567	.633	.700	.767	.833	1.00	1.20	1.40	1.60
21.0			.221	.257	.276	.294	.331	.349	.368	.404	.441	.478	.551	.625	.698	.772	.845	.919	1.10	1.32	1.54	1.76
22.0			.242	.282	.303	.323	.363	.383	.403	.444	.484	.524	.605	.686	.766	.847	.928	1.01	1.21	1.45	1.69	1.94
23.0				.309	.331	.353	.397	.419	.441	.485	.529	.573	.661	.749	.838	.926	1.01	1.10	1.32	1.59	1.85	2.12
24.0				.336	.360	.384	.432	.456	.480	.528	.576	.624	.720	.816	.912	1.01	1.10	1.20	1.44	1.73	2.02	2.30
25.0				.365	.391	.417	.469	.495	.521	.573	.625	.677	.781	.885	.990	1.09	1.20	1.30	1.56	1.88	2.19	2.50
26.0				.394	.423	.451	.507	.535	.563	.620	.676	.732	.845	.958	1.07	1.18	1.30	1.41	1.69	2.03	2.37	2.70
27.0					.456	.486	.547	.577	.608	.668	.729	.790	.911	1.03	1.15	1.28	1.40	1.52	1.82	2.19	2.55	2.92
28.0					.490	.523	.588	.621	.653	.719	.784	.849	.980	1.11	1.24	1.37	1.50	1.63	1.96	2.35	2.74	3.14
29.0					.526	.561	.631	.666	.701	.771	.841	.911	1.05	1.19	1.33	1.47	1.61	1.75	2.10	2.52	2.94	3.36
30.0					.563	.600	.675	.713	.750	.825	.900	.975	1.13	1.28	1.43	1.58	1.73	1.88	2.25	2.70	3.15	3.60
31.0						.641	.721	.761	.801	.881	.961	1.04	1.20	1.36	1.52	1.68	1.84	2.00	2.40	2.88	3.36	3.84
32.0						.683	.768	.811	.853	.939	1.02	1.11	1.28	1.45	1.62	1.79	1.96	2.13	2.56	3.07	3.58	4.10
33.0						.726	.817	.862	.908	.998	1.09	1.18	1.36	1.54	1.72	1.91	2.09	2.27	2.72	3.27	3.81	4.36
34.0						.771	.867	.915	.963	1.06	1.16	1.25	1.45	1.64	1.83	2.02	2.22	2.41	2.89	3.47	4.05	4.62
36.0							.972	1.03	1.08	1.19	1.30	1.40	1.62	1.84	2.05	2.27	2.48	2.70	3.24	3.89	4.54	5.18
38.0							1.08	1.14	1.20	1.32	1.44	1.56	1.81	2.05	2.29	2.53	2.77	3.01	3.61	4.33	5.05	5.78
40.0							1.20	1.27	1.33	1.47	1.60	1.73	2.00	2.27	2.53	2.80	3.07	3.33	4.00	4.80	5.60	6.40
42.0							1.32	1.40	1.47	1.62	1.76	1.91	2.21	2.50	2.79	3.09	3.38	3.68	4.41	5.29	6.17	7.06
44.0								1.53	1.61	1.77	1.94	2.10	2.42	2.74	3.07	3.39	3.71	4.03	4.84	5.81	6.78	7.74
46.0								1.68	1.76	1.94	2.12	2.29	2.65	3.00	3.35	3.70	4.06	4.41	5.29	6.35	7.41	8.46
48.0								1.82	1.92	2.11	2.30	2.50	2.88	3.26	3.65	4.03	4.42	4.80	5.76	6.91	8.06	9.22
50.0								1.98	2.08	2.29	2.50	2.71	3.13	3.54	3.96	4.38	4.79	5.21	6.25	7.50	8.75	10.0
52.0									2.25	2.48	2.70	2.93	3.38	3.83	4.28	4.73	5.18	5.63	6.76	8.11	9.46	10.8
54.0									2.43	2.67	2.92	3.16	3.65	4.13	4.62	5.10	5.59	6.08	7.29	8.75	10.2	11.7
56.0									2.61	2.87	3.14	3.40	3.92	4.44	4.97	5.49	6.01	6.53	7.84	9.41	11.0	12.5
58.0									2.80	3.08	3.36	3.64	4.21	4.77	5.33	5.89	6.45	7.01	8.41	10.1	11.8	13.5
60.0										3.30	3.60	3.90	4.50	5.10	5.70	6.30	6.90	7.50	9.00	10.8	12.6	14.4
64.0										3.75	4.10	4.44	5.12	5.80	6.49	7.17	7.85	8.53	10.2	12.3	14.3	16.4
68.0										4.24	4.62	5.01	5.78	6.55	7.32	8.09	8.86	9.63	11.6	13.9	16.2	18.5
72.0										4.75	5.18	5.62	6.48	7.34	8.21	9.07	9.94	10.8	13.0	15.6	18.1	20.7

For use of this Table see Flexure Examples 1 to 10, 16

SOURCE: DESIGN HANDBOOK
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STORMWATER DRAIN JUNCTION BOXES

TCC 2-29-12 HH 5-31-12

FLEXURE 1.2

Flexure 1.2—Coefficients for rectangular sections with compression reinforcement

$\rho = \omega \frac{f'_c}{f_y} = \frac{A_s}{bd}$

$K_u = \phi f'_c \omega (1 - .59\omega)$

$\frac{c}{d} = 1.18 \frac{\omega}{\beta_1}$

$a_u = \phi f_y (1 - .59\omega)/12{,}000$

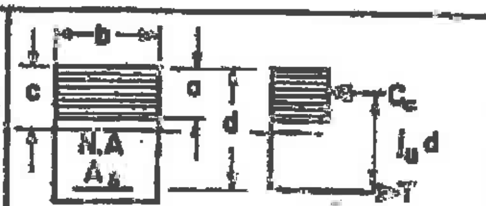
$\frac{a}{d} = \beta_1 \frac{c}{d}$

$A_s = \frac{M_u}{a_u d}$

$j_u = 1 - \frac{a}{2d}$

$\beta_1 = 0.85$

$F = \frac{M_u}{K_u}$



A_s in square inches and M_u is in ft - kips

ω	K_u	f'_c = 4,000								c/d	a/d	j_u
		f_y = 40,000		f_y = 50,000		f_y = 60,000		f_y = 80,000				
		ρ	a_u	ρ	a_u	ρ	a_u	ρ	a_u			
.020	71	.0020	2.96	.0016	3.71	.0013	4.65	.0010	5.93	.028	.024	.988
.030	106	.0030	2.93	.0024	3.68	.0020	4.42	.0015	5.89	.042	.035	.982
.040	141	.0040	2.93	.0032	3.66	.0027	4.39	.0020	5.86	.056	.047	.976
.050	175	.0050	2.91	.0040	3.64	.0033	4.37	.0025	5.82	.069	.059	.971
.060	208	.0060	2.89	.0048	3.62	.0040	4.34	.0030	5.79	.083	.071	.965
.070	242	.0070	2.88	.0056	3.60	.0047	4.31	.0035	5.75	.097	.083	.959
.080	274	.0080	2.86	.0064	3.57	.0053	4.29	.0040	5.72	.111	.094	.953
.090	307	.0090	2.84	.0072	3.55	.0060	4.26	.0045	5.68	.125	.106	.947
.100	339	.0100	2.82	.0080	3.53	.0067	4.23	.0050	5.65	.139	.118	.941
.110	370	.0110	2.81	.0088	3.51	.0073	4.21	.0055	5.61	.153	.130	.935
.120	401	.0120	2.79	.0096	3.48	.0080	4.18	.0060	5.58	.167	.142	.929
.130	432	.0130	2.77	.0104	3.46	.0087	4.15	.0065	5.54	.180	.153	.923
.140	462	.0140	2.75	.0112	3.44	.0093	4.13	.0070	5.50	.194	.165	.917
.150	492	.0150	2.73	.0120	3.42	.0100	4.10	.0075	5.47	.208	.177	.912
.160	522	.0160	2.72	.0128	3.40	.0107	4.08	.0080	5.43	.222	.189	.906
.170	551	.0170	2.70	.0136	3.37	.0113	4.05	.0085	5.40	.236	.201	.900
.180	579	.0180	2.68	.0144	3.35	.0120	4.02	.0090	5.36	.250	.212	.894
.190	607	.0190	2.66	.0152	3.33	.0127	4.00	.0095	5.33	.264	.224	.888
.200	635	.0200	2.65	.0160	3.31	.0133	3.97	.0100	5.29	.278	.236	.882
.210	662	.0210	2.63	.0168	3.29	.0140	3.94	.0105	5.26	.292	.248	.876
.220	689	.0220	2.61	.0176	3.26	.0147	3.92	.0110	5.22	.305	.260	.870
.230	716	.0230	2.59	.0184	3.24	.0153	3.89	.0115	5.19	.319	.271	.864
.240	742	.0240	2.58	.0192	3.22	.0160	3.86	.0120	5.15	.333	.283	.858
.250	767	.0250	2.56	.0200	3.20	.0167	3.84	.0125	5.12	.347	.295	.853
.260	792	.0260	2.54	.0208	3.17	.0173	3.81	.0130	5.08	.361	.307	.847
.270	817	.0270	2.52	.0216	3.15	.0180	3.78	.0135	5.04	.375	.319	.841
.280	841	.0280	2.50	.0224	3.13	.0187	3.76	.0140	5.01	.389	.330	.835
.290	865	.0290	2.49	.0232	3.11	.0193	3.73			.403	.342	.829
.300	889	.0300	2.47	.0240	3.09	.0200	3.70			.416	.354	.823
.310	912	.0310	2.45	.0248	3.06	.0207	3.68			.430	.366	.817
.320	935	.0320	2.43	.0256	3.04	.0213	3.65			.444	.378	.811
.330	957	.0330	2.42	.0264	3.02					.458	.389	.805
.340	978	.0340	2.40	.0272	3.00					.472	.401	.799
.350	1000	.0350	2.38							.486	.413	.794
.360	1021	.0360	2.36							.500	.425	.788
.370	1041	.0370	2.35							.514	.437	.782

1. When $h_f / d \geqq a/d$, beam is designed as rectangular section; when $h_f / d < a/d$, beam is designed as a T section (See Flexure 2).
2. Values of ρ above upper solid line are for percentages smaller than ρ min., ρ min. = $200/f_y$. See section 10.5, ACI 318-71.
3. Values of ρ below lower solid line would exceed ρ max. = 0.75 ρ_b. For values of ρ_{max} see values of (ρ_b - 0.75 ρ_b) max. in Flexure 2. See Section 10.3.2, ACI 318-71.
4. For deflection see Section 9.5 ACI 318-71 and "Deflection" chapter in this handbook.
5. Check for Crack Control where f_y > 40,000. See Section 10.6. ACI 318-71 and under chapter "Reinforcement" in this handbook.
6. The capacity reduction factor φ = 0.9 has been included in the table values.
7. For use of this Table, see Flexure Examples 1 to 10, 16, 18

SOURCE: DESIGN HANDBOOK

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STORMWATER DRAIN JUNCTION BOXES

TCL 2-29-12 HH 5-31-12

PJ. 7 of 21

Reinforcement 16—Areas of bars in a section 1 ft wide

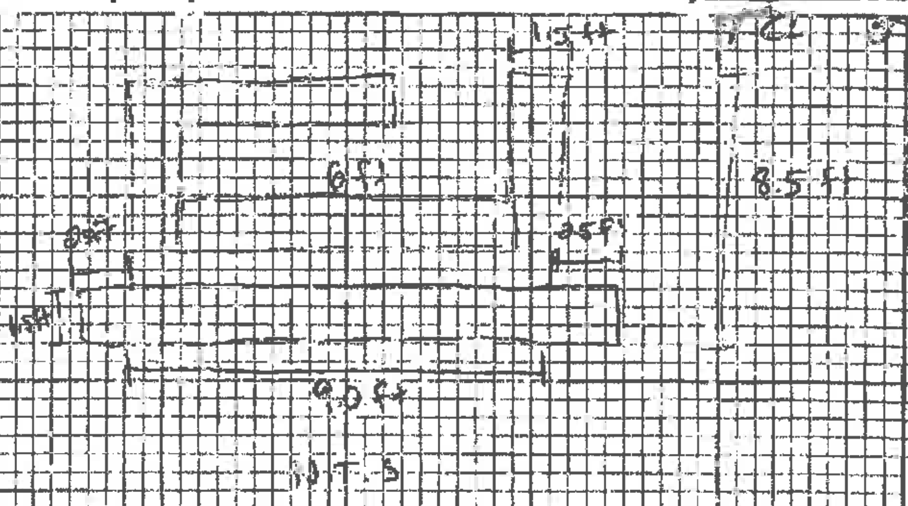
Areas A_s (or A'_s) [illegible]

Spacing	#3	#4	#5	#6	#7	#8	#9	#10	#11	#14	#18	Spacing
2	.66	1.20	1.86	[illegible]								2
2-½	.53	.96	1.49	2.11	2.88	[illegible]						2-½
3	.44	.80	1.24	1.76	2.40	3.16	4.00					3
3-½	.38	.69	1.06	1.51	2.06	2.71	3.43	4.35				3-½
4	.33	.60	.93	1.32	1.80	2.37	3.00	3.81	4.68			4
4-½	.29	.53	.83	1.17	1.60	2.11	2.67	3.39	4.16	6.00		4-½
5	.26	.48	.74	1.06	1.44	1.90	2.40	3.05	3.74	5.40	9.60	5
5-½	.24	.44	.68	.96	1.31	1.72	2.18	2.77	3.40	4.91	8.73	6-½
6	.22	.40	.62	.88	1.20	1.58	2.00	2.54	3.12	4.50	8.00	6
6-½	.20	.37	.57	.81	1.11	1.46	1.85	2.34	2.88	4.15	7.38	6-½
7	.19	.34	.53	.75	1.03	1.35	1.71	2.18	2.67	3.86	6.86	7
7-½	.18	.32	.50	.70	.96	1.26	1.60	2.03	2.50	3.60	6.40	7-½
8	.17	.30	.47	.66	.90	1.19	1.50	1.91	2.34	3.38	6.00	8
8-½	.16	.28	.44	.62	.85	1.12	1.41	1.79	2.20	3.18	5.65	8-½
9	.15	.27	.41	.59	.80	1.05	1.33	1.69	2.08	3.00	5.33	9
9-½	.14	.25	.39	.56	.76	1.00	1.26	1.60	1.97	2.84	5.05	9-½
10	.13	.24	.37	.53	.72	.95	1.20	1.52	1.87	2.70	4.80	10
10-½	.13	.23	.35	.50	.69	.90	1.14	1.45	1.78	2.57	4.57	10-½
11	.12	.22	.34	.48	.65	.86	1.09	1.39	1.70	2.45	4.36	11
11-½	.11	.21	.32	.46	.63	.82	1.04	1.33	1.63	2.35	4.17	11-½
12	.11	.20	.31	.44	.60	.79	1.00	1.27	1.56	2.25	4.00	12
13		.18	.29	.41	.55	.73	.92	1.17	1.44	2.08	3.69	13
14		.17	.27	.38	.51	.68	.86	1.09	1.34	1.93	3.43	14
15		.16	.25	.35	.48	.63	.80	1.02	1.25	1.80	3.20	15
16		.15	.23	.33	.45	.59	.75	.95	1.17	1.69	3.00	16
17		.14	.22	.31	.42	.56	.71	.90	1.10	1.59	2.82	17
18		.13	.21	.29	.40	.53	.67	.85	1.04	1.50	2.67	18

SOURCE: DESIGN HANDBOOK
ACI PUBLICATION SP-17 (73)

STORMWATER DRAIN JUNCTION BOXES

Project: AWI Project No.:
Subject: Floatation calc JB-2 Ex Method Sheet No. 8 of 22
Drawing No.:
Computed by: MT Date: 3-2-12 Checked by: MH Date: 5-30-12



$F_{uplift} = PA = \gamma h A$

$\gamma_{water} = 62.4\ lb/ft^3$ $\gamma_{concrete} = 150\ lb/ft^3$

$\gamma_{soil} = 120\ lb/ft^3$

$$F_{uplift} = \gamma_{water} hA = \frac{(62.4\ lb/ft^3)(8.5\ ft)(2.5+2.5+9\ ft^2)}{1000\ lbs/kip} = 7.4\ kips \quad \boxed{Eqn.\ 10}$$

$$F_{soil} = \gamma_{soil} hA = \left[\frac{(120\ lb/ft^3)(7\ ft)(2.5\ ft^2)}{1000\ lbs/kip}\right] \cdot 2 = 4.2\ kips \quad \boxed{Eqn.\ 11}$$

$$F_{concrete} = \gamma_{concrete} hA = \frac{(150\ lb/ft^3)(1.5\ ft)(2.5+2.5+9\ ft^2)}{1000\ lbs/kip} = 3.15\ kips \quad \boxed{Eqn.\ 12}$$

$$F_{concrete\ wall} = \gamma_{concrete} hA = \frac{(150\ lb/ft^3)(7\ ft)(1.5\ ft^2)}{1000\ lbs/kip} = 3.15\ kips \quad \boxed{Eqn.\ 13}$$

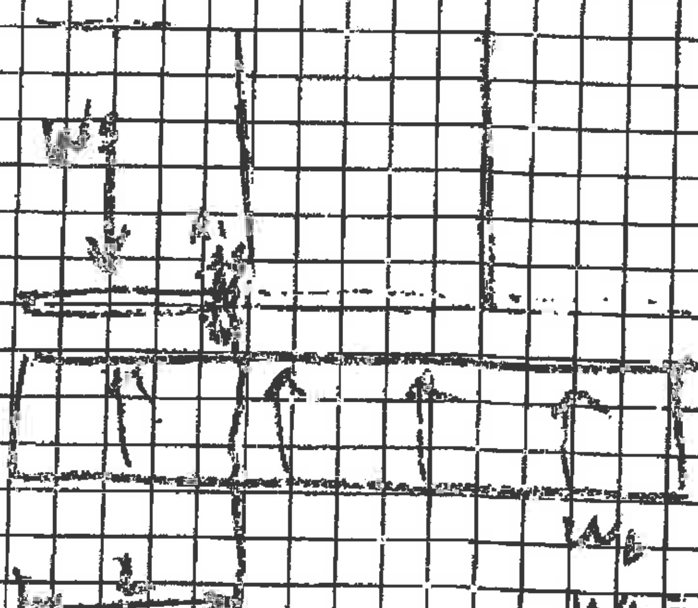
$$F_{down} = F_{soil} + F_{concrete} + F_{concrete\ wall} = 4.2 + 3.15 + 3.15 = 10.5\ kips \quad \boxed{Eqn.\ 14}$$

$F_{down} > F_{uplift}$

10.5 kips > 7.4 kips ✓

Project AWI
Subject Outside slab Top Steel
STORMWATER DRAIN JUNCTION BOXES
Project No.
Sheet No. 9 of 22
Drawing No.
Computed by TCC Date 2-29-12 Checked by MJ / FH Date 5-30-12 / 5-31-12

FOOTING



W_b BEARING PRESSURE

W_b = UPLIFT − WEIGHT

OUTSIDE SLAB TOP STEEL

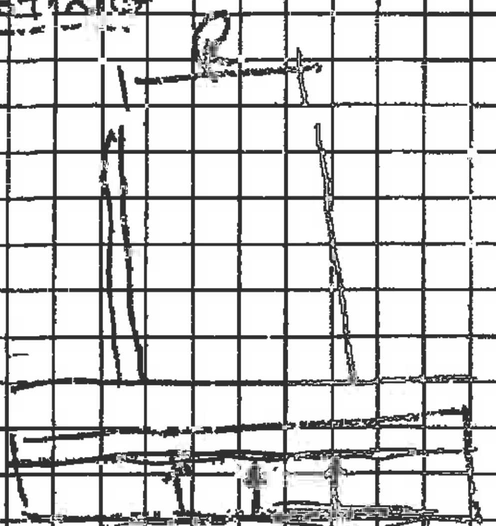
$$M_1 = \left(W_s \times \frac{L}{2}\right) - \left(W_b \times L \times \frac{L}{2}\right) \qquad \boxed{\text{Eqn 15}}$$

$$B_1 = W_s \downarrow - W_b \times L \uparrow$$

STORMWATER DRAIN JUNCTION BOXES

Project AWI | Project No.
Subject Footing center top and wall bottom | Sheet No. 10 of 22
Drawing No.
Computed by TCC Date 2-29-12 Checked by MT Date 5-30-12
HH 5-31-12

FOOTING



M_{max} @ end $= \frac{w\ell^2}{12}$ (BOTTOM) $\boxed{\text{Eqn. 16}}$

M @ center $= \frac{w\ell^2}{24}$ (TOP) $\boxed{\text{Eqn. 17}}$

NOTE: See Reference Drawing Attached

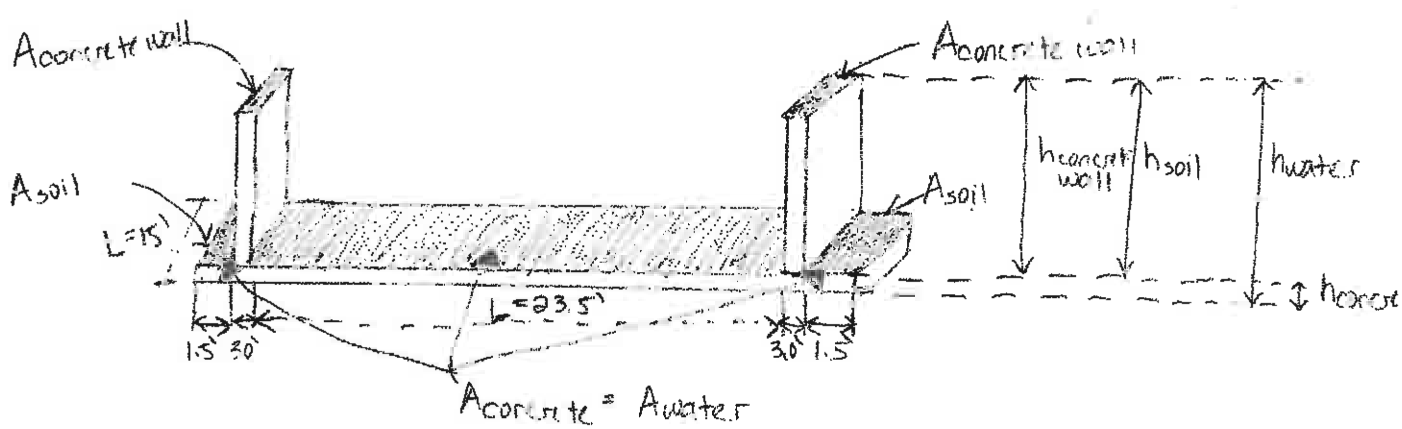
Floatation Calculation

Junction Box 1

AWI

Parameter	Value	Unit	
γ_{water}	62.4	lb/ft^3	
γ_{soil}	120	lb/ft^3	
$\gamma_{concrete}$	120	lb/ft^3	
h_{water}	17.3	ft	
A_{water}	487.5	ft^2	
F_{uplift}	527.3	kips	Eqn. 10
h_{soil}	14.3	ft	
A_{soil}	124.5	ft^2	width = 2.5 ft
F_{soil}	214.14	kips	Eqn. 11
$h_{concrete}$	3	ft	
$A_{concrete}$	487.5	ft^2	= 15 ft x 32.5 ft
$F_{concrete}$	175.50	kips	Eqn. 12
$h_{concrete\ wall}$	14.33333	ft	
$A_{concrete\ wall}$	213	ft^2	
$F_{concrete\ wall}$	366.36	kips	Eqn. 13
F_{down}	756.00	kips	Eqn. 14
F_{uplift}	527.3	kips	
F.S.	1.4		→ conservative assumptions

1) Groundwater table is at ground level (worst case)
2) Weight of roof not included



All excel calculations at spreadsheet file:

\\warwick\warwickfp\Projects\AWI\Calcs Rev 6-20-12.xlsx

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Rebar Sizing Calculation
Junction Box 1
AWI

PREPARER: MT 3-2-12
CHECKER: HH 5-30-12
APPROVER: TC 5-31-12
All SPREADSHEETS

ROOF CONDITION 1

Parameter	Value	Unit	Description/Notes
W_c	80	lb/in^2	HS-20 loading, see attached sheet
W_c	11,520	lb/ft^2	
L	6	ft	Shortest span between JB1 sides
M_{end}	35	kip-ft	Eqn. 1, max moment at ends
M_{center}	17	kip-ft	Eqn. 2, max moment at center

ROOF CONDITION 2

Parameter	Value	Unit	Description/Notes
P	16000	lb	HS-20 wheel load, see attached sheet
M_{max}	2.0	kip-ft	Eqn. 3, max moment at center and ends

WALL CONDITION

Parameter	Value	Unit	Description/Notes
h	8	ft	Height of box (not including footing)
b	2.67	ft	1/3h
a	5.33	ft	2/3h
W_c	80	lb/in^2	HS-20 loading, see attached sheet
P_1	1.5	kip	Resultant of 5% of HS-20 load uniformly distributed over height of wall
P_2	14.4	kip	Resultant of remaining HS-20 load distributed over height of wall
ρ_w	62.4	lb/ft^3	Density of water
ρ_s	57.6	lb/ft^3	Density of soil
k	0.33		Coefficient for soil force
P_w	2.00	kip-ft	Force of water
P_s	0.61	kip-ft	Force of soil
R_1	12.1	kip-ft	Eqn. 4, resultant force at top of wall
R_2	6.44	kip-ft	Eqn. 5, resultant force at bottom of wall
M_1	19.7	kip-ft	Eqn. 6, resultant moment at top of wall
M_2	12.7	kip-ft	Eqn. 7, resultant moment at bottom of wall

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	2.0	kip-ft	Applied design moment from Eqn. 3
M_u	24.0	kip-in	
K_u	122.4	kip/in^2	Eqn. 8
p	0.0033		From Flexure 1.2 Table
a_u	3.78		From Flexure 1.2 Table
A_s	0.55		Eqn. 9, area of steel
#6 at 9" spacing			From Reinforcement 16 table

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	32	in	Thickness of concrete minus 4 in.
F_u	1.02	in^3	From Flexure 5 Table
M_u	19.7	kip-ft	Applied design moment from Eqn. 6
M_u	236.1	kip-in	
K_u	231.5	kip/in^2	Eqn. 8
p	0.0047		From Flexure 1.2 Table
a_u	4.31		From Flexure 1.2 Table
A_s	1.80		Eqn. 9, area of steel
#10 bars at 8" spacing			From Reinforcement 16 table

Rebar Sizing Calculation
Junction Box 1
AWI

FOOTING outside slab top steel

Parameter	Value	Unit	Description/Notes
h	9.5	ft	Height of box (including footing)
l	3	ft	Overhang length
ρ_s	120	lb/ft^3	Density of soil
W_s	3.42	kips	Weight of soil
F_{down}	15.06	kips	From floatation calculation
F_{uplift}	10.296	kips	From floatation calculation
l	15	ft	Bottom slab length
W_b	0.3176	lb/ft	Bearing pressure
M	3.70	kip-ft	Eqn. 15, resultant moment

FOOTING center; worst case no water

Parameter	Value	Unit	Description/Notes
l	6	ft	Clear span length of footing
W_b	1.004	kip/ft	Bearing pressure
M_{max}	3.0	kip-ft	Eqn. 16, moment at end
M_{center}	1.5	kip-ft	Eqn. 17, moment at center

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	3.70	kip-ft	Applied design moment
M_u	44.4	kip-in	
K_u	226.6	kip/in^2	Eqn. 8
ρ	0.0047		From Flexure 1.2 Table
a_u	4.31		From Flexure 1.2 Table
A_s	0.79		Eqn. 9, area of steel
#8 bars at 9" spacing			From Reinforcement 16 table

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	3.0	kip-ft	Applied design moment from Eqn. 1
M_u	36.1	kip-in	
K_u	184.4	kip/in^2	Eqn. 8
ρ	0.004		From Flexure 1.2 Table
a_u	4.37		From Flexure 1.2 Table
A_s	0.67		Eqn. 9, area of steel
#8 bars at 9" spacing			From Reinforcement 16 table

NOTE: See Reference Drawings Attached

Floatation Calculation
Junction Box 2
AWI

Parameter	Value	Unit	
γ_{water}	62.4	lb/ft^3	
γ_{soil}	120	lb/ft^3	
$\gamma_{concrete}$	120	lb/ft^3	
h_{water}	11	ft	
A_{water}	450	ft^2	
F_{uplift}	308.9	kips	Eqn. 10
h_{soil}	9	ft	
A_{soil}	200	ft^2	width = 2.5 ft
F_{soil}	216	kips	Eqn. 11
$h_{concrete}$	2	ft	
$A_{concrete}$	450	ft^2	= 15 ft x 30 ft
$F_{concrete}$	108.00	kips	Eqn. 12
$h_{concrete\ wall}$	9	ft	
$A_{concrete\ wall}$	144	ft^2	
$F_{concrete\ wall}$	155.52	kips	Eqn. 13
F_{down}	479.52	kips	Eqn. 14
F_{uplift}	308.9	kips	
F.S.	1.6		

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Rebar Sizing Calculation
Junction Box 2
AWI

ROOF CONDITION 1

Parameter	Value	Unit	Description/Notes
W_c	80	lb/in^2	HS-20 loading, see attached sheet
W_c	11,520	lb/ft^2	
L	6	ft	Shortest span between JB2 sides
M_{end}	35	kip-ft	Eqn. 1, max moment at ends
M_{center}	17	kip-ft	Eqn. 2, max moment at center

ROOF CONDITION 2

Parameter	Value	Unit	Description/Notes
P	16000	lb	HS-20 wheel load, see attached sheet
M_{max}	2.0	kip-ft	Eqn. 3, max moment at center and ends

WALL CONDITION

Parameter	Value	Unit	Description/Notes
h	5.5	ft	Height of box (not including footing)
b	1.83	ft	1/3h
a	3.67	ft	2/3h
W_c	80	lb/in^2	HS-20 loading, see attached sheet
P_1	1.0	kip	Resultant of 5% of HS-20 load uniformly distributed over height of wall
P_2	9.9	kip	Resultant of remaining HS-20 load distributed over height of wall
ρ_w	62.4	lb/ft^3	Density of water
ρ_s	57.6	lb/ft^3	Density of soil
k	0.33		Coefficient for soil force
P_w	0.94	kip	Force of water
P_s	0.29	kip	Force of soil
R_1	8.2	kip	Eqn. 4, resultant force at top of wall
R_2	4.0	kip	Eqn. 5, resultant force at bottom of wall
M_1	9.1	kip-ft	Eqn. 6, resultant moment at top of wall
M_2	5.5	kip-ft	Eqn. 7, resultant moment at bottom of wall

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	2.0	kip-ft	Applied design moment from **Eqn. 3**
M_u	24.0	kip-in	
K_u	122.4	kip/in^2	Eqn. 8
ρ	0.0033		From Flexure 1.2 Table
a_u	4.21		From Flexure 1.2 Table
A_s	0.55		Eqn. 9, area of steel
#6 at 9" spacing			From **Reinforcement 16 table**

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	20	in	Thickness of concrete minus 4 in.
F_u	0.4	in^3	From Flexure 5 Table
M_u	9.1	kip-ft	Applied design moment from Eqn. 6
M_u	108.9	kip-in	
K_u	272.2	kip/in^2	Eqn. 8
ρ	0.0053		From Flexure 1.2 Table
a_u	4.15		From Flexure 1.2 Table
A_s	1.27		Eqn. 9, area of steel
#8 at 7" spacing			From Reinforcement 16 table

Rebar Sizing Calculation
Junction Box 2
AWI

FOOTING outside slab top steel

Parameter	Value	Unit	Description/Notes
h	8.5	ft	Height of box (including footing)
l	3	ft	overhang length
ρ_s	120	lb/ft^3	Density of soil
W_s	3.06	kips	Weight of soil
F_{down}	11.78	kips	From floatation calculation
F_{uplift}	7.956	kips	From floatation calculation
l	15	ft	Bottom slab length
W_b	0.2546	lb/ft	Bearing pressure
M	3.44	kip-ft	Eqn. 15, resultant moment

FOOTING center, worst case no water

Parameter	Value	Unit	Description/Notes
l	6	ft	Clear span length of footing
W_b	0.785	kip/ft	Bearing pressure
M_{max}	2.4	kip-ft	Eqn. 16, moment at end
M_{center}	1.2	kip-ft	Eqn. 17, moment at center

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	3.44	kip-ft	Applied design moment
M_u	41.3	kip-in	
K_u	210.9	kip/in^2	Eqn. 8
ρ	0.004		From Flexure 1.2 Table
a_u	4.31		From Flexure 1.2 Table
A_s	0.67		Eqn. 9, area of steel
#8 bars at 9" spacing			From Reinforcement 16 Table

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	2.4	kip-ft	Applied design moment from Eqn. 1
M_u	28.3	kip-in	
K_u	144.2	kip/in^2	Eqn. 8
ρ	0.0033		From Flexure 1.2 Table
a_u	4.37		From Flexure 1.2 Table
A_s	0.55		Eqn. 9, area of steel
#8 bars at 9" spacing			From Reinforcement 16 Table

NOTE: See Reference Drawings Attached

Floatation Calculation

Junction Box 3

AWI

Parameter	Value	Unit	
γ_{water}	62.4	lb/ft^3	
γ_{soil}	120	lb/ft^3	
$\gamma_{concrete}$	120	lb/ft^3	
h_{water}	8	ft	
A_{water}	576	ft^2	
F_{uplift}	287.5	kips	Eqn. 10
h_{soil}	6.5	ft	
A_{soil}	252	ft^2	width = 3 ft
F_{soil}	196.56	kips	Eqn. 11
$h_{concrete}$	1.5	ft	
$A_{concrete}$	576	ft^2	= 24 ft x 24 ft
$F_{concrete}$	103.68	kips	Eqn. 12
$h_{concrete\ wall}$	6.5	ft	
$A_{concrete\ wall}$	117	ft^2	
$F_{concrete\ wall}$	91.26	kips	Eqn. 13
F_{down}	391.50	kips	Eqn. 14
F_{uplift}	287.5	kips	
F.S.	1.4		

Rebar Sizing Calculation
Junction Box 3
AWI

ROOF CONDITION 1

Parameter	Value	Unit	Description/Notes
W_c	80	lb/in^2	HS-20 loading, see attached sheet
W_c	11,520	lb/ft^2	
L	15	ft	Shortest span between JB3 sides
M_{end}	216	kip-ft	Eqn. 1, max moment at ends
M_{center}	108	kip-ft	Eqn. 2, max moment at center

ROOF CONDITION 2

Parameter	Value	Unit	Description/Notes
P	16000	lb	HS-20 wheel load, see attached sheet
M_{max}	2.0	kip-ft	Eqn. 3, max moment at center and ends

WALL CONDITION

Parameter	Value	Unit	Description/Notes
h	4	ft	Height of box (not including footing)
b	1.33	ft	1/3h
a	2.67	ft	2/3h
W_c	80	lb/in^2	HS-20 loading, see attached sheet
P_1	0.8	kip	Resultant of 5% of HS-20 load uniformly distributed over height of wall
P_2	7.2	kip	Resultant of remaining HS-20 load distributed over height of wall
ρ_w	62.4	lb/ft^3	Density of water
ρ_s	57.6	lb/ft^3	Density of soil
k	0.33		Coefficient for soil force
P_w	0.50	kip	Force of water
P_s	0.15	kip	Force of soil
R_1	5.9	kip	Eqn. 4, resultant force at top of wall
R_2	2.7	kip	Eqn. 5, resultant force at bottom of wall
M_1	4.7	kip-ft	Eqn. 6, resultant moment at top of wall
M_2	2.8	kip-ft	Eqn. 7, resultant moment at bottom of wall

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	2.0	kip-ft	Applied design moment from Eqn. 3
M_u	24.0	kip-in	
K_u	122.4	kip/in^2	Eqn. 8
ρ	0.0033		From Flexure 1.2 Table
a_u	4.21		From Flexure 1.2 Table
A_s	0.55		Eqn. 9, area of steel
#6 at 9" spacing			From Reinforcement 16 table

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	4.7	kip-ft	Applied design moment from Eqn. 6
M_u	56.7	kip-in	
K_u	289.4	kip/in^2	Eqn. 8
ρ	0.006		From Flexure 1.2 Table
a_u	4.15		From Flexure 1.2 Table
A_s	1.01		Eqn. 9, area of steel
#8 at 7" spacing			From Reinforcement 16 table

Rebar Sizing Calculation
Junction Box 3
AWI

FOOTING outside slab top steel

Parameter	Value	Unit	Description/Notes
h	5.5	ft	Height of box (including footing)
l	3	ft	overhang length
ρ_s	120	lb/ft^3	Density of soil
W_s	1.98	kips	Weight of soil
F_{down}	12.06	kips	From floatation calculation
F_{uplift}	6.552	kips	From floatation calculation
l	15	ft	Bottom slab length
W_b	0.3672	lb/ft	Bearing pressure
M	1.32	kip-ft	Eqn. 15, resultant moment

FOOTING center; worst case no water

Parameter	Value	Unit	Description/Notes
l	15	ft	Clear span length of footing
W_b	0.804	kip/ft	Bearing pressure
M_{max}	15.1	kip-ft	Eqn. 16, moment at end
M_{center}	7.5	kip-ft	Eqn. 17, moment at center

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	26	in	Thickness of concrete minus 4 in.
F_u	0.676	in^3	From Flexure 5 Table
M_u	1.32	kip-ft	Applied design moment
M_u	15.8	kip-in	
K_u	23.4	kip/in^2	Eqn. 8
ρ	0.0033		From Flexure 1.2 Table
a_u	4.31		From Flexure 1.2 Table
A_s	1.03		Eqn. 9, area of steel
#10 bars at 9" spacing			From Reinforcement 16 Table

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	26	in	Thickness of concrete minus 4 in.
F_u	0.676	in^3	From Flexure 5 Table
M_u	15.1	kip-ft	Applied design moment from Eqn. 1
M_u	180.9	kip-in	
K_u	267.6	kip/in^2	Eqn. 8
ρ	0.0053		From Flexure 1.2 Table
a_u	4.37		From Flexure 1.2 Table
A_s	1.65		Eqn. 9, area of steel
#10 bars at 9" spacing			From Reinforcement 16 Table

NOTE: See Reference Drawings Attached

Floatation Calculation
Junction Box 4
AWI

Parameter	Value	Unit	
γ_{water}	62.4	lb/ft^3	
γ_{soil}	120	lb/ft^3	
$\gamma_{concrete}$	120	lb/ft^3	
h_{water}	7.5	ft	
A_{water}	360	ft^2	
F_{uplift}	168.5	kips	Eqn. 10
h_{soil}	6	ft	
A_{soil}	198	ft^2	width = 3 ft
F_{soil}	142.56	kips	Eqn. 11
$h_{concrete}$	1.5	ft	
$A_{concrete}$	360	ft^2	= 15 ft x 24 ft
$F_{concrete}$	64.80	kips	Eqn. 12
$h_{concrete\ wall}$	6	ft	
$A_{concrete\ wall}$	90	ft^2	
$F_{concrete\ wall}$	64.80	kips	Eqn. 13
F_{down}	272.16	kips	Eqn. 14
F_{uplift}	168.5	kips	
F.S.	1.6		

Rebar Sizing Calculation
Junction Box 4
AWI

ROOF CONDITION 1

Parameter	Value	Unit	Description/Notes
W_c	80	lb/in^2	HS-20 loading, see attached sheet
W_c	11,520	lb/ft^2	
L	6	ft	Shortest span between JB4 sides
M_{end}	35	kip-ft	Eqn. 1, max moment at ends
M_{center}	17	kip-ft	Eqn. 2, max moment at center

ROOF CONDITION 2

Parameter	Value	Unit	Description/Notes
P	16000	lb	HS-20 wheel load, see attached sheet
M_{max}	2.0	kip-ft	Eqn. 3, max moment at center and ends

WALL CONDITION

Parameter	Value	Unit	Description/Notes
h	3.5	ft	Height of box (not including footing)
b	1.17	ft	1/3h
a	2.33	ft	2/3h
W_c	80	lb/in^2	HS-20 loading, see attached sheet
P_1	0.7	kip	Resultant of 5% of HS-20 load uniformly distributed over height of wall
P_2	6.3	kip	Resultant of remaining HS-20 load distributed over height of wall
ρ_w	62.4	lb/ft^3	Density of water
ρ_s	57.6	lb/ft^3	Density of soil
k	0.33		Coefficient for soil force
P_w	0.38	kip	Force of water
P_s	0.12	kip	Force of soil
R_1	5.1	kip	Eqn. 4, resultant force at top of wall
R_2	2.3	kip	Eqn. 5, resultant force at bottom of wall
M_1	3.6	kip-ft	Eqn. 6, resultant moment at top of wall
M_2	2.1	kip-ft	Eqn. 7, resultant moment at bottom of wall

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	2.0	kip-ft	Applied design moment from Eqn. 3
M_u	24.0	kip-in	
K_u	122.4	kip/in^2	Eqn. 8
ρ	0.0033		From Flexure 1.2 Table
a_u	4.21		From Flexure 1.2 Table
A_s	0.55		Eqn. 9, area of steel
#6 at 9" spacing			From Reinforcement 16 table

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	3.6	kip-ft	Applied design moment from Eqn. 6
M_u	43.2	kip-in	
K_u	220.4	kip/in^2	Eqn. 8
ρ	0.0047		From Flexure 1.2 Table
a_u	4.15		From Flexure 1.2 Table
A_s	0.79		Eqn. 9, area of steel
#8 at 7" spacing			From Reinforcement 16 table

Rebar Sizing Calculation
Junction Box 4
AWI

FOOTING outside slab top steel

Parameter	Value	Unit	Description/Notes
h	5	ft	Height of box (including footing)
l	3	ft	Overhang length
ρ_s	120	lb/ft^3	Density of soil
W_s	1.8	kips	Weight of soil
F_{down}	9.23	kips	From floatation calculation
F_{uplift}	6.084	kips	From floatation calculation
l	15	ft	Bottom slab length
W_b	0.2094	lb/ft	Bearing pressure
M	1.76	kip-ft	Eqn. 15, resultant moment

FOOTING center, worst case no water

Parameter	Value	Unit	Description/Notes
l	6	ft	Clear span length of footing
W_b	0.615	kip/ft	Bearing pressure
M_{max}	1.8	kip-ft	Eqn. 16, moment at end
M_{center}	0.9	kip-ft	Eqn. 17, moment at center

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	1.76	kip-ft	Applied design moment
M_u	21.1	kip-in	
K_u	107.6	kip/in^2	Eqn. 8
ρ	0.0033		From Flexure 1.2 Table
a_u	4.31		From Flexure 1.2 Table
A_s	0.55		Eqn. 9, area of steel
#8 bars at 9" spacing			From Reinforcement 16 Table

REBAR SIZING

Parameter	Value	Unit	Description/Notes
b	12	in	Width of compressive area
d	14	in	Thickness of concrete minus 4 in.
F_u	0.196	in^3	From Flexure 5 Table
M_u	1.8	kip-ft	Applied design moment from Eqn. 1
M_u	22.1	kip-in	
K_u	113.0	kip/in^2	Eqn. 8
ρ	0.0033		From Flexure 1.2 Table
a_u	4.37		From Flexure 1.2 Table
A_s	0.55		Eqn. 9, area of steel
#8 bars at 9" spacing			From Reinforcement 16 Table

References

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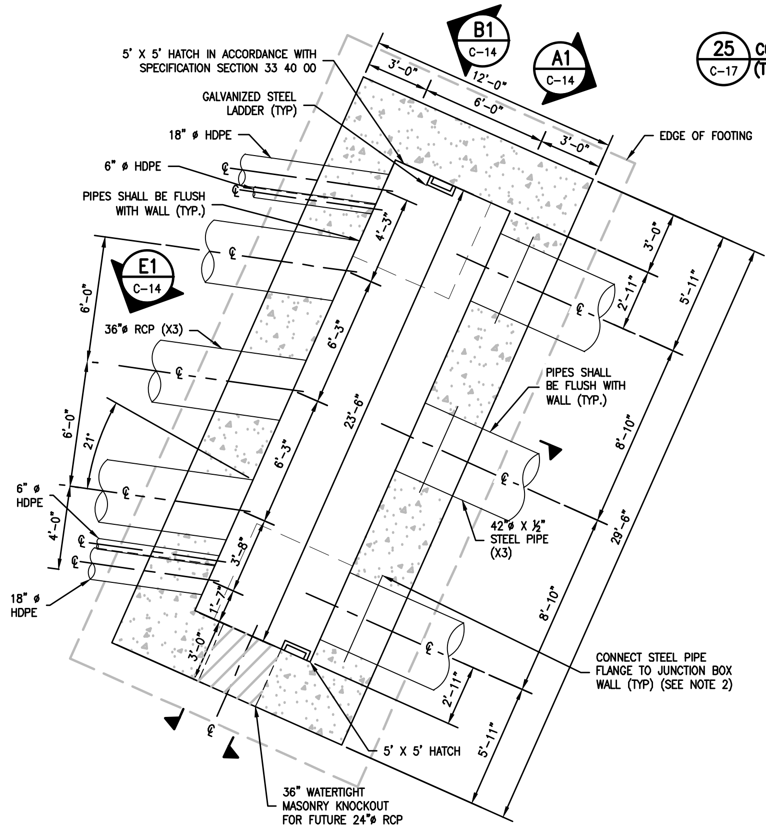
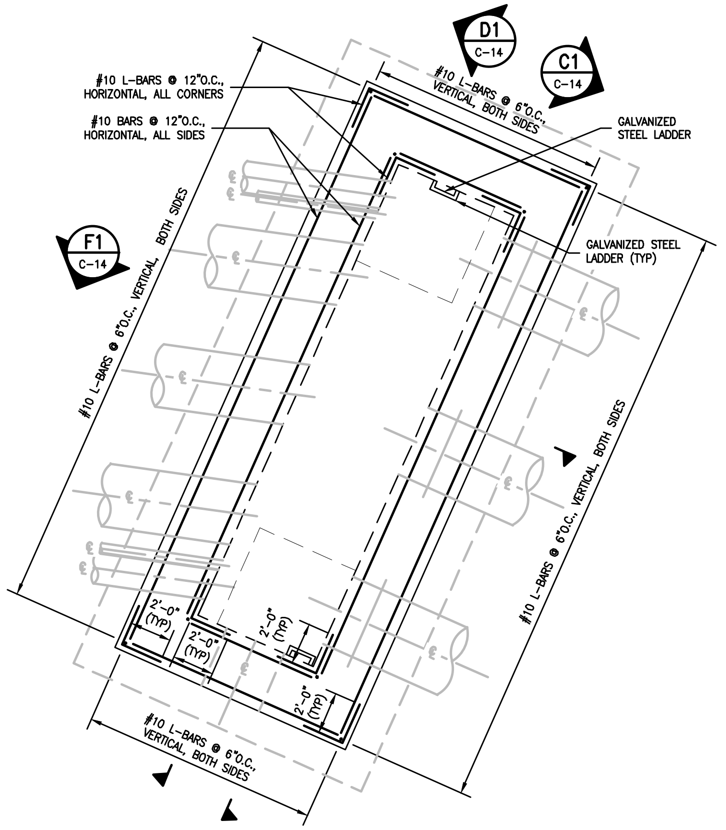
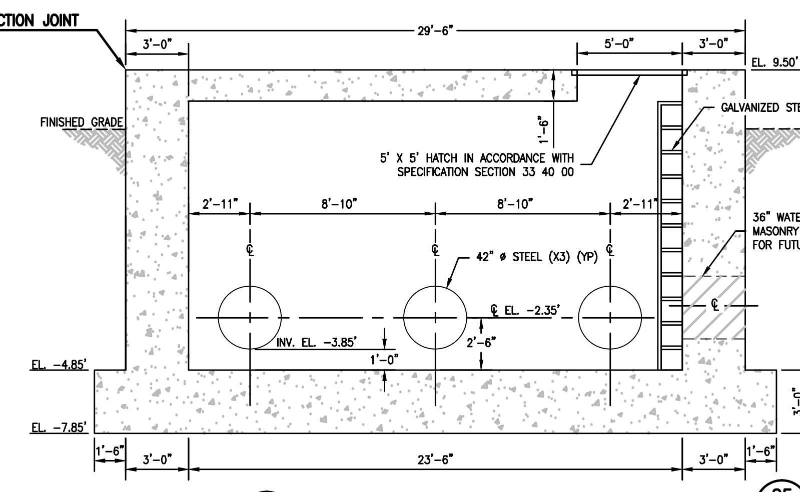


PLAN



REINFORCEMENT PLAN

21 SD JUNCTION BOX 1 (C-5)
SCALE: 1/4"=1'



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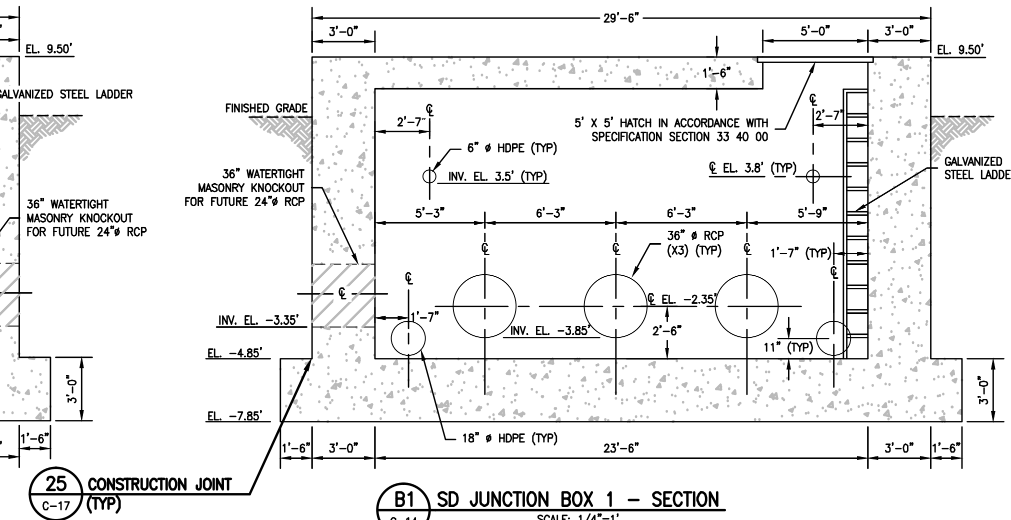

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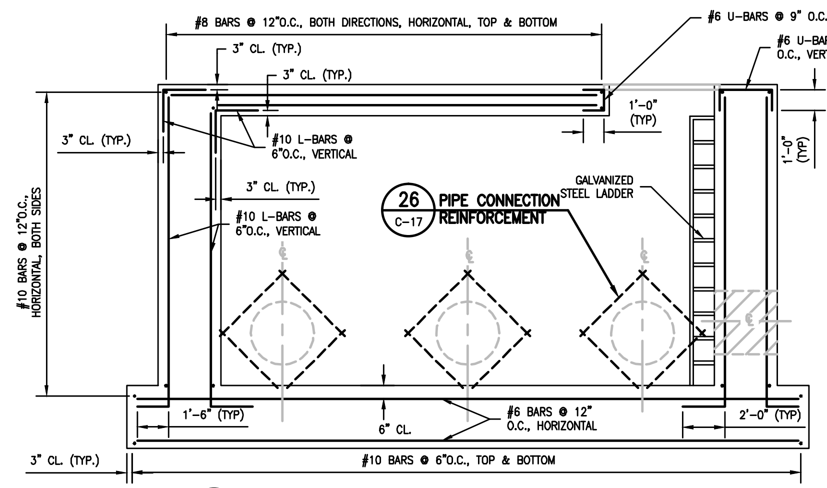

C1 SD JUNCTION BOX 1 – REINFORCEMENT SECTION (C-14)
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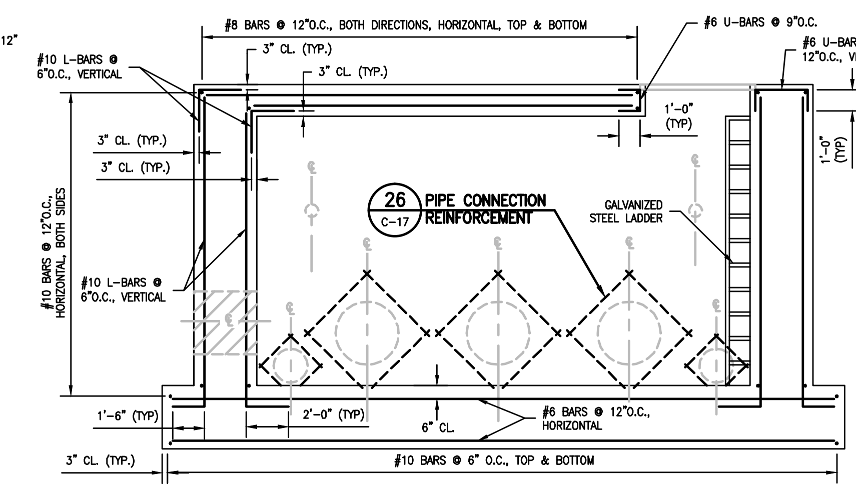
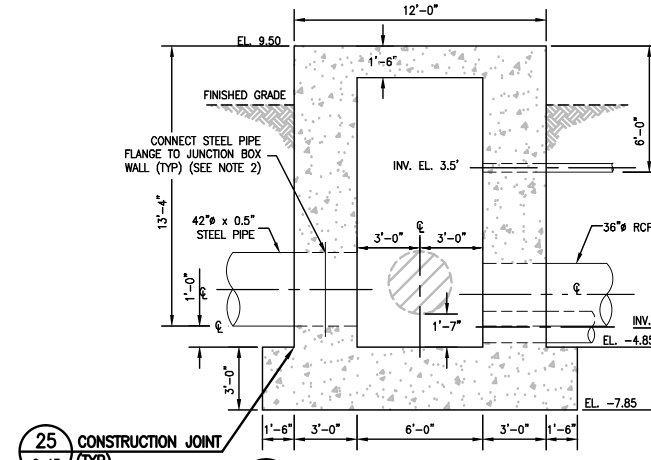


D1 SD JUNCTION BOX 1 – REINFORCEMENT SECTION (C-14)
SCALE: 1/4"=1'



E1 SD JUNCTION BOX 1 – SECTION (C-14)
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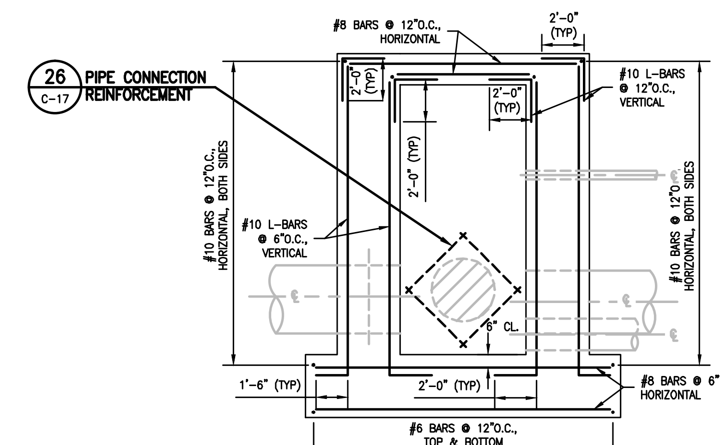

F1 SD JUNCTION BOX 1 – REINFORCEMENT SECTION (C-14)
SCALE: 1/4"=1'

NOTES:

1. CONTRACTOR TO SELECT EITHER PRECAST OR CAST-IN-PLACE METHOD OF CONSTRUCTION FOR SD JUNCTION BOXES. CONSTRUCTION SHALL BE IN ACCORDANCE WITH REQUIREMENTS SPECIFIED IN SECTION 03 00 00 CONCRETE.
2. CONTRACTOR SHALL CAST STEEL PIPE FLANGE INTO JUNCTION BOX WALL FOR CAST-IN-PLACE CONSTRUCTION. CONTRACTOR SHALL BOLT STEEL PIPE FLANGE TO INSIDE FACE OF STORM DRAIN JUNCTION BOX FOR PRECAST JUNCTION BOX CONSTRUCTION.
3. DIAGONAL REINFORCEMENT LOCATED IN LONGITUDINAL WALL. SHOWN FOR INFORMATION ONLY. (SEE DETAIL 26 – PIPE CONNECTION REINFORCEMENT ON C-17)
4. SEE DRAWING C-4 FOR PLAN VIEW OF STORM DRAIN JUNCTION BOX. SEE DRAWING C-5 FOR PLAN AND PROFILE VIEW OF STORM DRAIN JUNCTION BOX.

NO.	DATE	BY	REVISIONS DESCRIPTION

SEAL

REMEDIAL DESIGN
ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT
ATLANTIC WOOD INDUSTRIES SUPERFUND SITE
PORTSMOUTH, VIRGINIA

STORM DRAIN JUNCTION BOX 1 DETAILS

PREPARED FOR: EPA

EA ENGINEERING, SCIENCE, AND TECHNOLOGY
Loveton Center
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(410) 771-4950

DATE	JULY 2012
DESIGNED BY	JLL
DRAWN BY	JAP
CHECKED BY	GAT
PROJECT MANAGER	PAP
PROJECT NUMBER	14530.11
DRAWING NUMBER	C-14
SHEET NUMBER	16 OF 22

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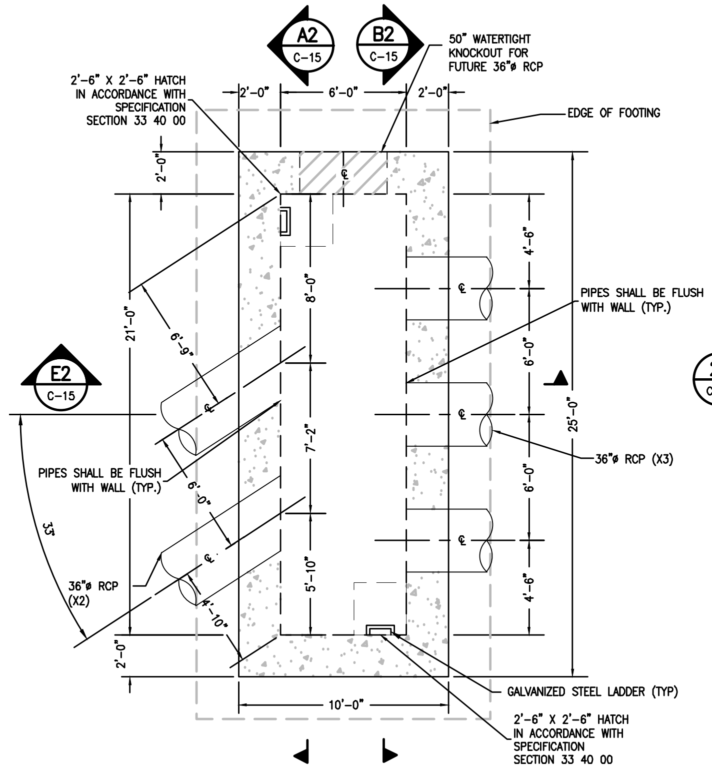
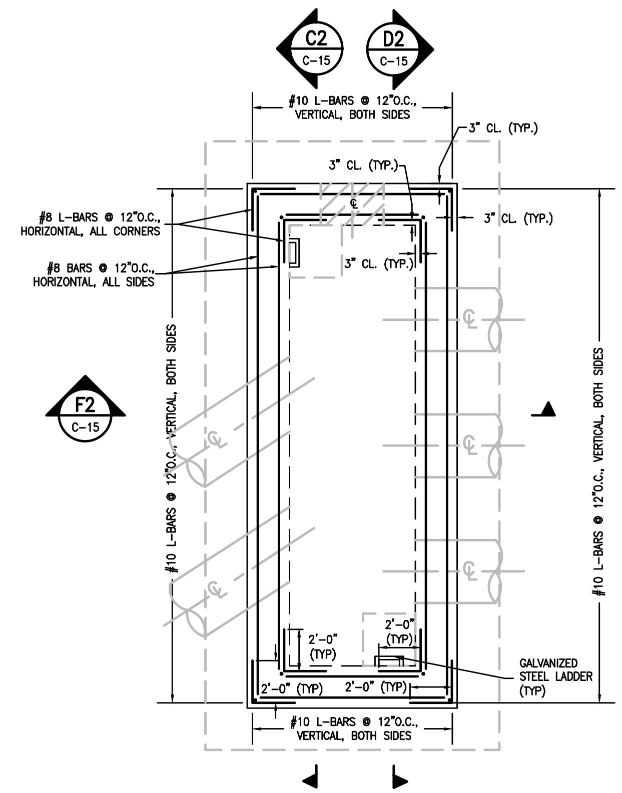
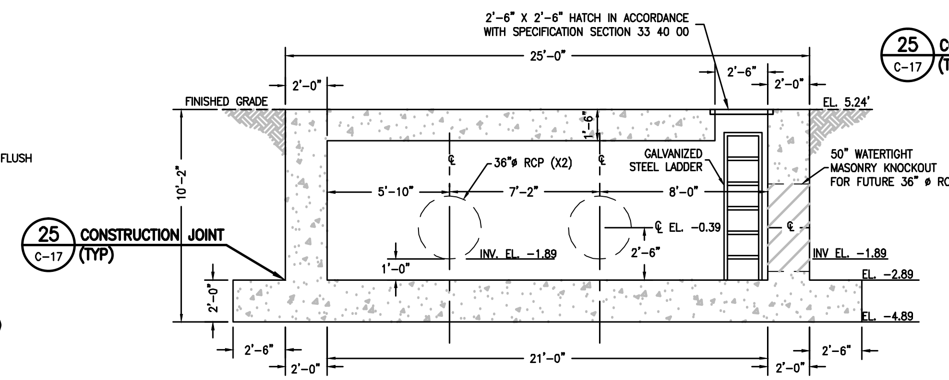


PLAN



REINFORCEMENT PLAN

22 C-5 C-6 SD JUNCTION BOX 2
SCALE: 1/4"=1'



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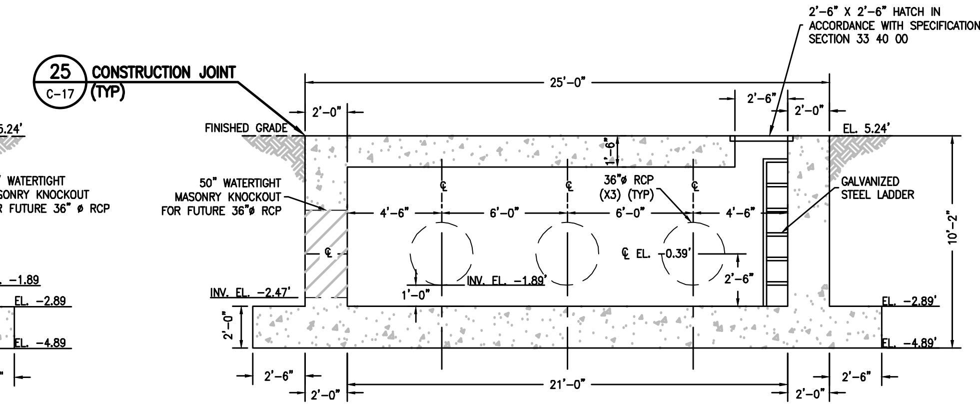

B2 C-15 SD JUNCTION BOX 2 – SECTION
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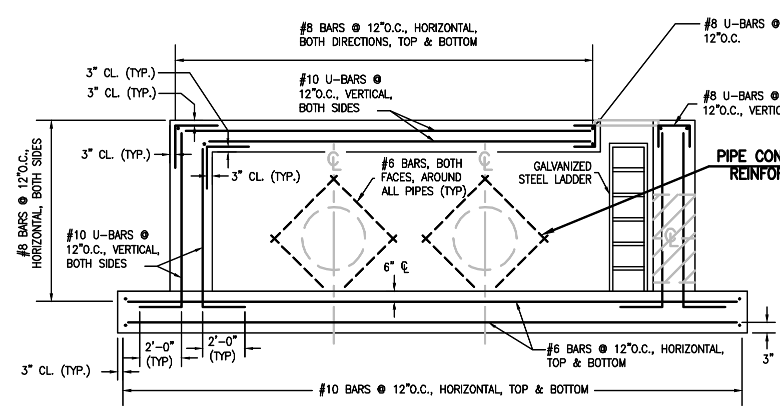

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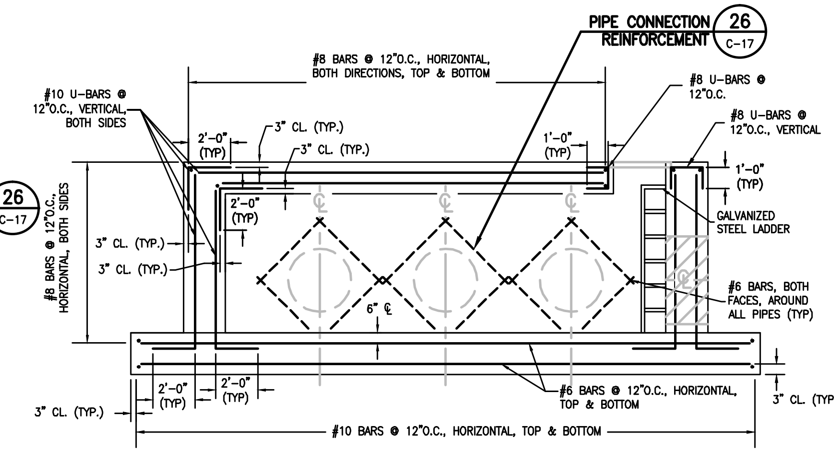

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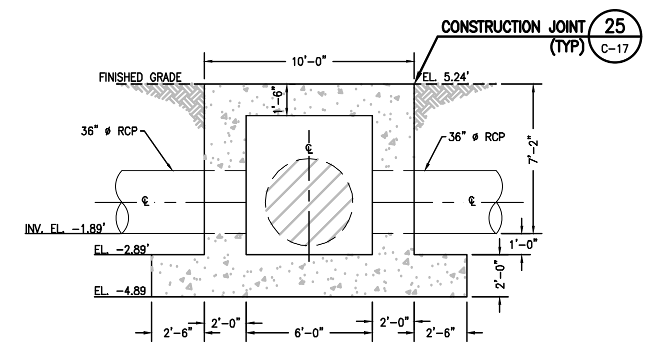
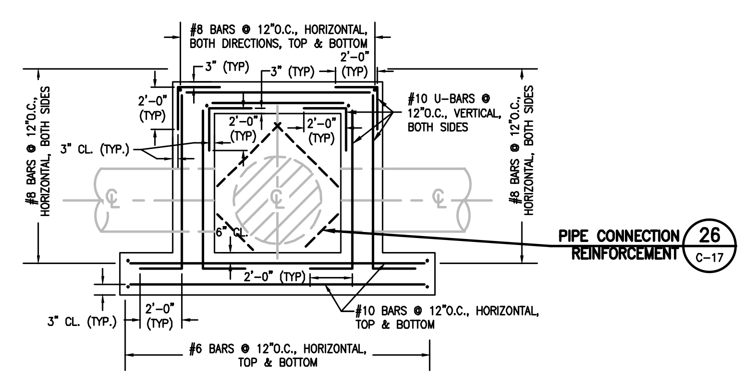


E2 C-15 SD JUNCTION BOX 2 – SECTION
SCALE: 1/4"=1'



F2 C-15 SD JUNCTION BOX 2 – REINFORCEMENT SECTION
SCALE: 1/4"=1'

NOTES:

1. CONTRACTOR TO SELECT EITHER PRECAST OR CAST-IN-PLACE METHOD OF CONSTRUCTION FOR SD JUNCTION BOXES. CONSTRUCTION SHALL BE IN ACCORDANCE WITH REQUIREMENTS SPECIFIED IN SECTION 03 00 00 CONCRETE.
2. DIAGONAL REINFORCEMENT LOCATED IN LONGITUDINAL WALL. SHOWN FOR INFORMATION ONLY. (SEE DETAIL 26 – PIPE CONNECTION REINFORCEMENT ON C-17)
3. SEE DRAWING C-4 FOR PLAN VIEW OF STORM JUNCTION BOX. SEE DRAWING C-5 FOR PLAN AND PROFILE VIEW OF STORM DRAIN JUNCTION BOX.

REVISIONS: NO. | DATE | BY | DESCRIPTION

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REMEDIAL DESIGN
ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT
ATLANTIC WOOD INDUSTRIES SUPERFUND SITE
PORTSMOUTH, VIRGINIA

STORM DRAIN JUNCTION BOX 2 DETAILS

PREPARED FOR:
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EA ENGINEERING, SCIENCE, AND TECHNOLOGY
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DATE	JULY 2012
DESIGNED BY	JLL
DRAWN BY	JAP
CHECKED BY	GAT
PROJECT MANAGER	PAP
PROJECT NUMBER	14530.11
DRAWING NUMBER	C-15
SHEET NUMBER	17 OF 22

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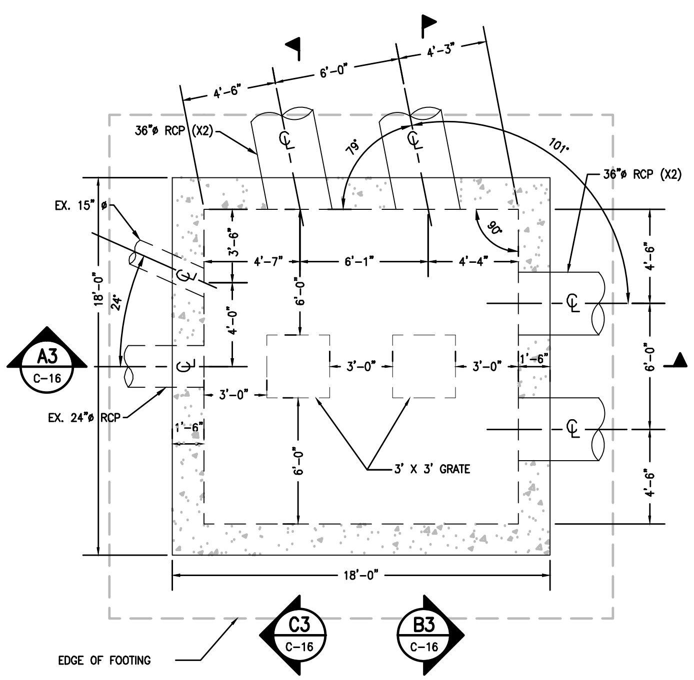
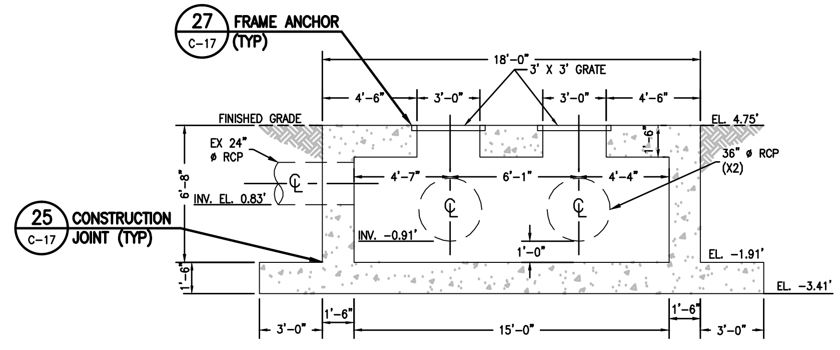
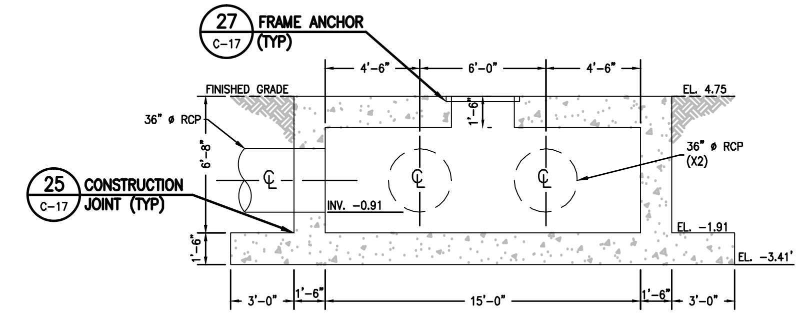


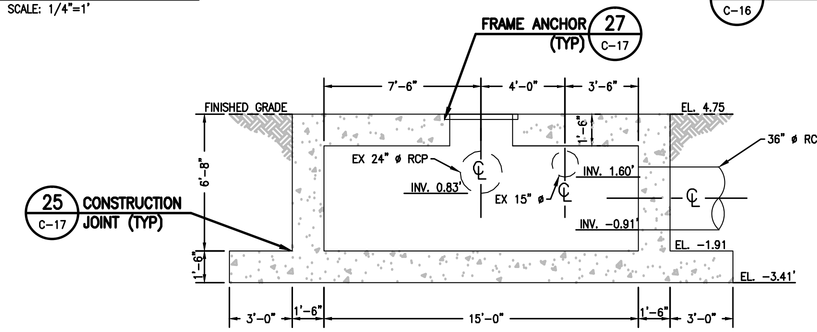
PLAN



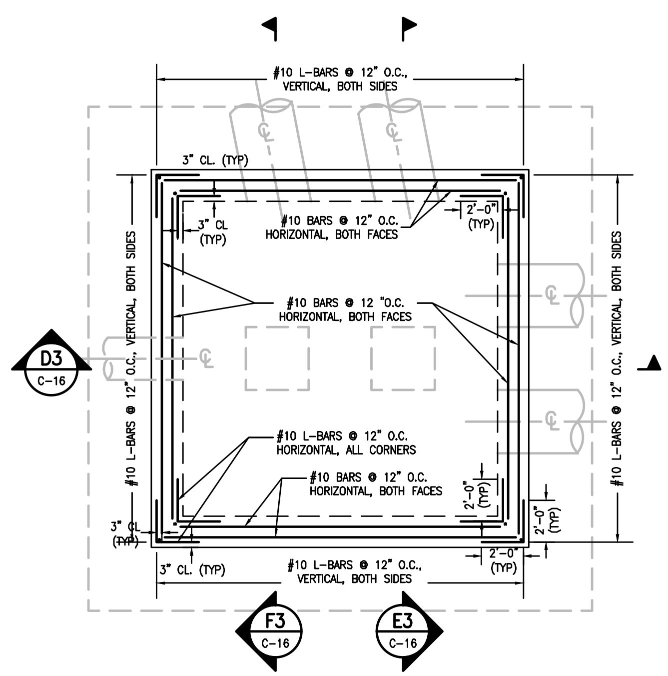
A3 SD JUNCTION BOX 3 – SECTION C-16
SCALE: 1/4"=1'



B3 SD JUNCTION BOX 3 – SECTION C-16
SCALE: 1/4"=1'

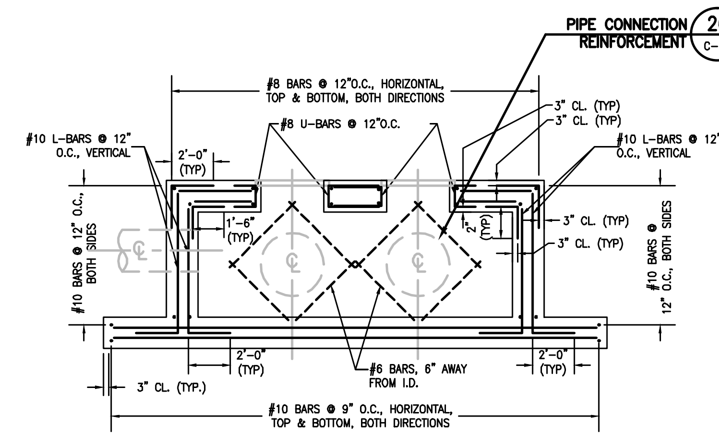


C3 SD JUNCTION BOX 3 – SECTION C-16
SCALE: 1/4"=1'



REINFORCEMENT PLAN

23 SD JUNCTION BOX 3 C-6
SCALE: 1/4"=1'



D3 SD JUNCTION BOX 3 – REINFORCEMENT SECTION C-16
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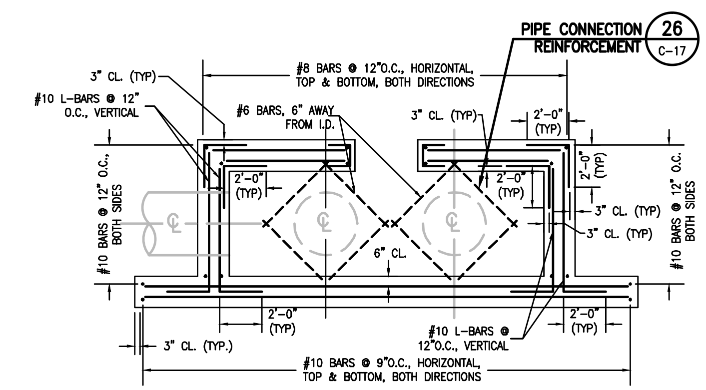
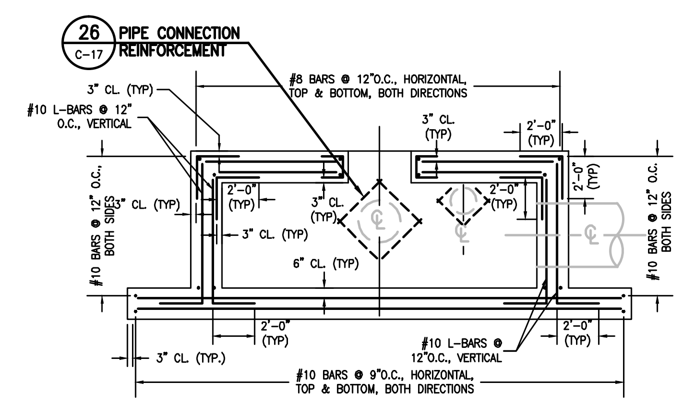


E3 SD JUNCTION BOX 3 – REINFORCEMENT SECTION C-16
SCALE: 1/4"=1'



F3 SD JUNCTION BOX 3 – REINFORCEMENT SECTION C-16
SCALE: 1/4"=1'

NOTES:

1. CONTRACTOR TO SELECT EITHER PRECAST OR CAST-IN-PLACE METHOD OF CONSTRUCTION FOR SD JUNCTION BOXES. CONSTRUCTION SHALL BE IN ACCORDANCE WITH REQUIREMENTS SPECIFIED IN SECTION 03 00 00 CONCRETE.
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3. SEE DRAWING C-4 FOR PLAN VIEW OF STORM JUNCTION BOX. SEE DRAWING C-5 FOR PLAN AND PROFILE VIEW OF STORM DRAIN JUNCTION BOX.

REVISIONS: NO. | DATE | BY | DESCRIPTION

SEAL

REMEDIAL DESIGN
ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT
ATLANTIC WOOD INDUSTRIES SUPERFUND SITE
PORTSMOUTH, VIRGINIA

STORM DRAIN JUNCTION BOX 3 DETAILS

PREPARED FOR:
EPA

EA ENGINEERING, SCIENCE, AND TECHNOLOGY
Loveton Center
15 Loveton Circle
Sparks, Maryland 21152
(410) 771-4950

DATE	JULY 2012
DESIGNED BY	JLL
DRAWN BY	JAP
CHECKED BY	GAT
PROJECT MANAGER	PAP
PROJECT NUMBER	14530.11
DRAWING NUMBER	C-16
SHEET NUMBER	18 OF 22

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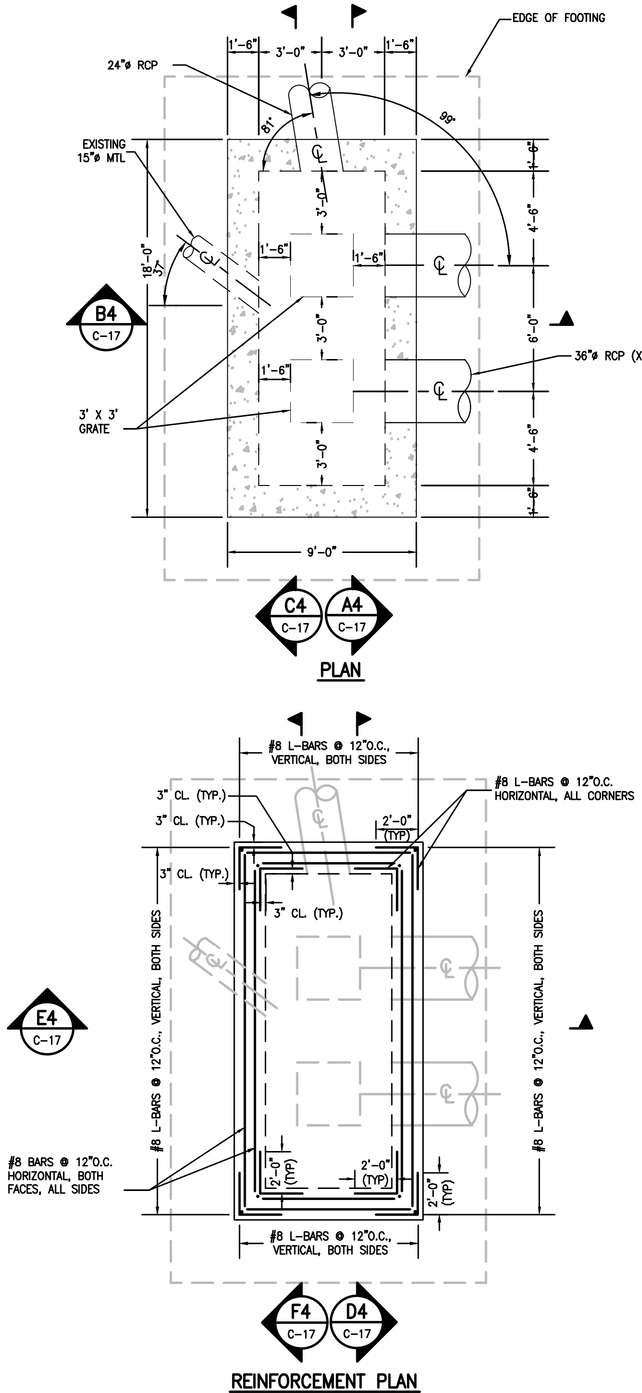
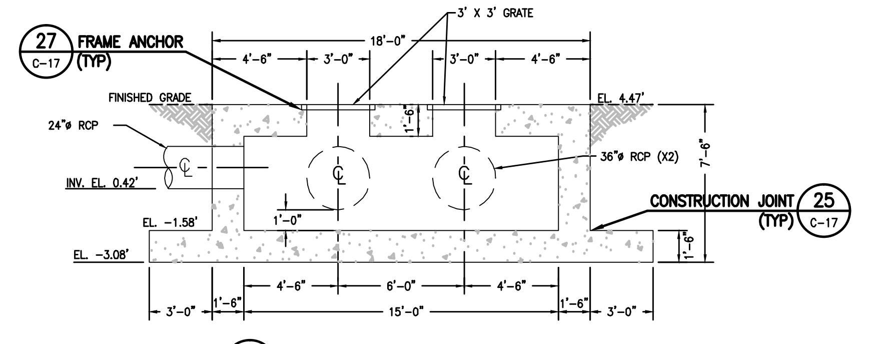


PLAN

REINFORCEMENT PLAN

24 SD JUNCTION BOX 4
C-6
SCALE: 1/4"=1'



A4 SD JUNCTION BOX 4 – SECTION
C-17
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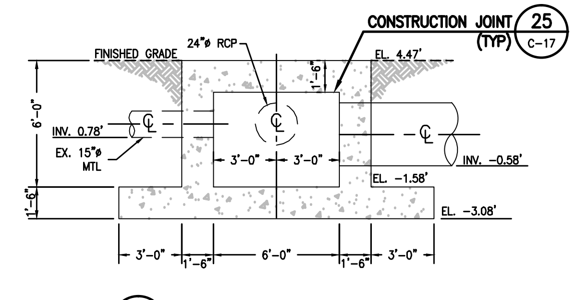

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C-17
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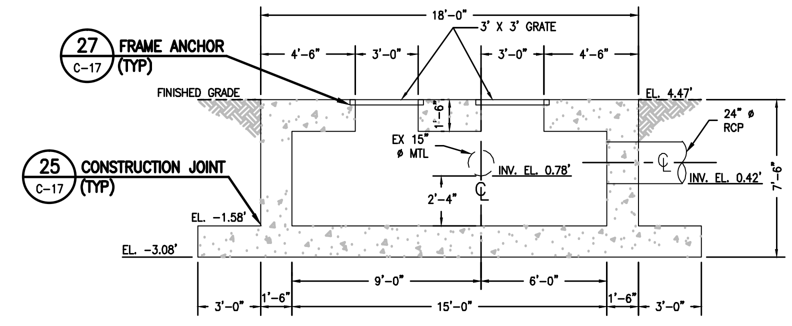

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C-17
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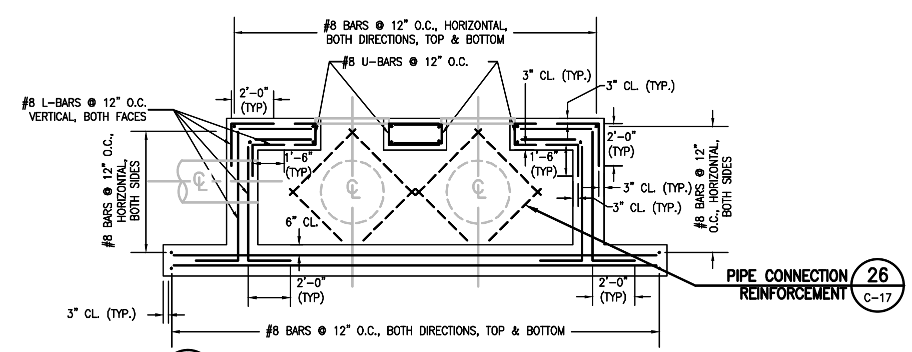

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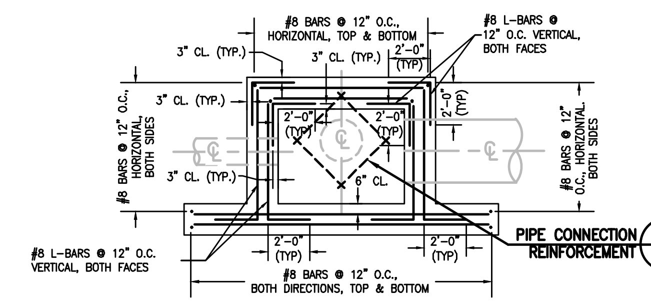

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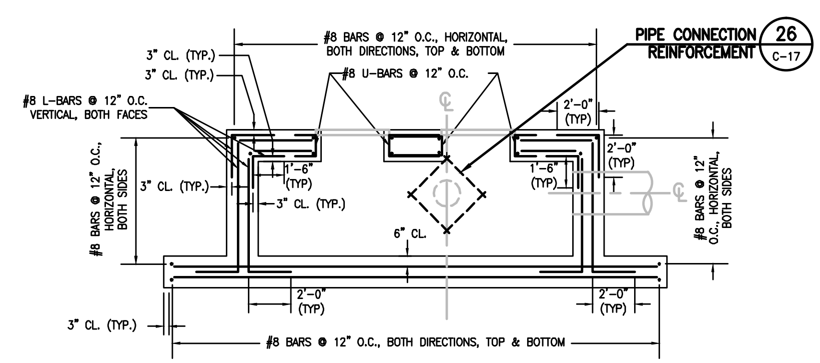

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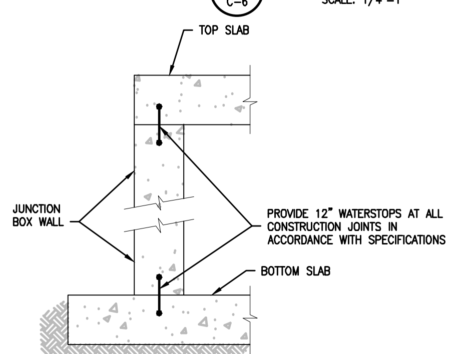
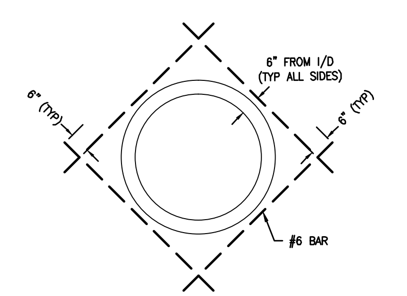
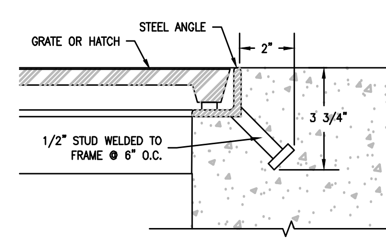


25 CONSTRUCTION JOINT
C-14
C-15, C-16, C-17



26 PIPE CONNECTION REINFORCEMENT
C-14
C-15, C-16, C-17



27 FRAME ANCHOR
C-16
C-17

NOTES:

1. CONTRACTOR TO SELECT EITHER PRECAST OR CAST-IN-PLACE METHOD OF CONSTRUCTION FOR SD JUNCTION BOXES. CONSTRUCTION SHALL BE IN ACCORDANCE WITH REQUIREMENTS SPECIFIED IN SECTION 03 00 00 CONCRETE.
2. DIAGONAL REINFORCEMENT LOCATED IN LONGITUDINAL WALL. SHOWN FOR INFORMATION ONLY. (SEE DETAIL 26 – PIPE CONNECTION REINFORCEMENT ON THIS SHEET)
3. SEE DRAWING C-4 FOR PLAN VIEW OF STORM JUNCTION BOX. SEE DRAWING C-5 FOR PLAN AND PROFILE VIEW OF STORM DRAIN JUNCTION BOX.

NO.	DATE	BY	REVISIONS DESCRIPTION

SEAL

REMEDIAL DESIGN
ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT
ATLANTIC WOOD INDUSTRIES SUPERFUND SITE
PORTSMOUTH, VIRGINIA

STORM DRAIN JUNCTION BOX 4 DETAILS

PREPARED FOR:

EPA

EA ENGINEERING, SCIENCE, AND TECHNOLOGY
Loveton Center
15 Loveton Circle
Sparks, Maryland 21152
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DATE	JULY 2012
DESIGNED BY	JLL
DRAWN BY	JAP
CHECKED BY	GAT
PROJECT MANAGER	PAP
PROJECT NUMBER	14530.11
DRAWING NUMBER	C-17
SHEET NUMBER	19 OF 22

Storm Drain Calculations
Storm Drain Loading Calculations

OBJECTIVE:

Determine the pipe class for the 36 inch RCP being installed as part of the Elm Avenue Storm Drain Relocation project on the AWI property. The pipe shall withstand live loads generated by large gantry cranes utilized by property owner.

GIVENS AND ASSUMPTIONS:

American Concrete Pipe Association (ACPA) Design Manual

The pipe will be installed using Type 1 Bedding. (ACPA)
The trench width is beyond the transition width; therefore, eliminating the transfer of load from the load prism to the surrounding undisturbed earth, thus the pipe will encounter higher loads and is considered to be installed in an Embankment Condition. (see sheet 4 and 5)
The minimum pipe cover is approximately 4 feet.
Crane weight equals 160,000 lbs.
Crane lift capacity equals 200,000 lbs.
Load per set of tires equals 90,000 lbs. (200,000 + 160,000)/4
Earth Bearing Pressure equals 120 pounds per square inch (psi) – from crane manufacturer (see sheet 6 and 7)
90,000 ÷ 120 = 750 sq. inches (assume 60 inches x 12.5 inches)
Average Weight per cubic foot of soil equals 110 pcf.

PROCEDURE:

Use the Indirect Design Method (Marston-Spangler method) to determine the 0.01-inch crack D-load strength. Utilize the following steps to determine the required strength:

1. **Earth Load**
   Due to the width of the trench the pipe will be installed in an embankment condition imposing additional load to the prism of soil therefore a Vertical Arching Factor will be applied to the prism load.

$$W_e = VAF \times PL$$

Where:
W_e = Earth Load (*lb/ft*)
VAF = Vertical Arching Factor = 1.35 for Type 1 Bedding
PL = Prism Load (*lb/ft*) (see sketch on sheet 4)

$$PL = w\left[H + \frac{D_o(4-\pi)}{8}\right]D_o$$

Where:

$w =$ soil weight (*lbs/ft*^{3})

$H =$ height of fill (*ft*)

$D_o =$ outside diameter of pipe (*ft*) (4 inch wall thickness)

Solved:

$$PL = 110\left[4 + \frac{3.67(4-\pi)}{8}\right]3.67 = 1773\ lb/ft$$

$$W_e = 1.35 \times 1773 = 2395\ lb/ft$$

2. Live Load

$$W_L = D_o \times P$$

Where:

$W_L =$ Live Load (*lb/ft*)

$D_o =$ outside diameter of pipe (*ft*) (4 inch wall thickness)

$P =$ Distributed Pressure

American Association of State Highway and Transportation Officials (AASHTO), Load and Resistance Factor Design (*LRFD) Bridge Design Specification*

$$P = \frac{90{,}000}{\left(\frac{12.5}{12} + LLDFxH\right)\left(\frac{60}{12} + LLDFxH\right)}$$

Where:

$LLDF =$ Live Load Distribution Factor = 1.75 from LRFD

$H =$ height of fill (ft)

Solved:

$$P = \frac{90{,}000}{(1.04+7)(5+7)} = 933\ lb/ft$$

$$W_L = 3.67 \times 933 = 3423\ lb/ft$$

American Concrete Pipe Association (ACPA) Design Manual Chapter 4

3. Selection of Pipe Strength

$$D-load = \left[\frac{W_e}{B_{fE}} + \frac{W_L}{B_{fLL}}\right] x \frac{F.S.}{D}$$

Where:

$D-load$ = Critical-Collapse Pressure (psi);

B_{fE} = Embankment Dead Load Bedding Factor = 4.0

B_{fLL} = Live Load Bedding Factor = 2.2

$F.S.$ = Factor of Safety = 1.0 for calculation of 0.01 crack D-load strength

D = pipe diameter (ft)

Solved:

$$D-load = \left[\frac{2395}{4.0} + \frac{3423}{2.2}\right] x \frac{1.0}{3} = 718\ lbs/ft/ft$$

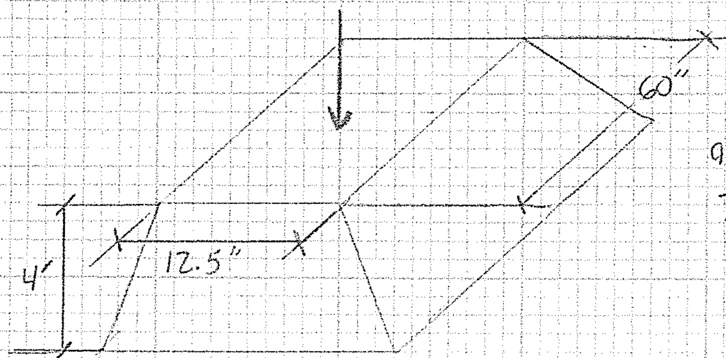
Based on ASTM C 76 the reinforced concrete pipe used in this project must have pipe strength of 718 lbs/ft/ft (4.99 lbs/in^2). Concrete used in ASTM C 76 reinforced concrete pipe has a minimum compressive strength of 4,000 psi.

CONCLUSIONS:

Based on site conditions and the required pipe strength, ASTM C 76 Class 3 reinforced concrete pipe has been specified to be used for the Elm Avenue Storm Drain Relocation.

Project ATLANTIC WOOD INDUSTRIES SUPERFUND SITE Project No. 1453011
Subject STORM DRAIN PIPE LOADING DESIGN Sheet No. 4 of 7
Drawing No.
Computed by JLL Date 6/4/12 Checked by GAT Date 6/26/12

PRISM LOAD (PL):



90000 lbs ÷ 120 lbs/in²
750 in² = 60" x 12.5"

LIVE LOAD DISTRIBUTION FACTOR - 1.75

DETERMINE TRANSITION WIDTH USING FIGURE 5 - TRANSITION WIDTHS FOR TYPE 1 INSTALLATIONS USING SAND AND GRAVEL BACKFILL

TRENCH SECTION

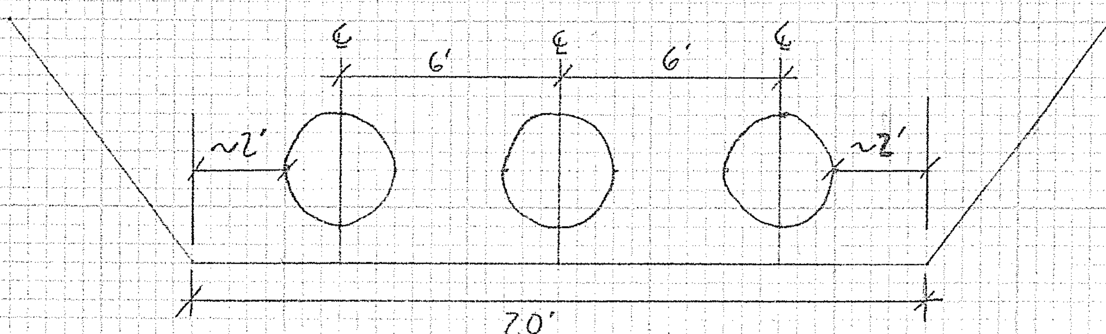


FIGURE 5 (SHEET 5 OF 5) USING A 4' DEPTH FOR 36 Ø RCP THE TRANSITION WIDTH < 6' ∴ PIPE INSTALLED IN EMBANKMENT CONDITION

EA 0289B 7/13/89

ATLANTIC WOOD SUPERFUND SITE
STORM DRAIN PIPE LOADING DESIGN

1453011
SHEET 5 OF 7

Figure 5 Transition Widths for Type 1 Installations Using Sand and Gravel Backfill Material

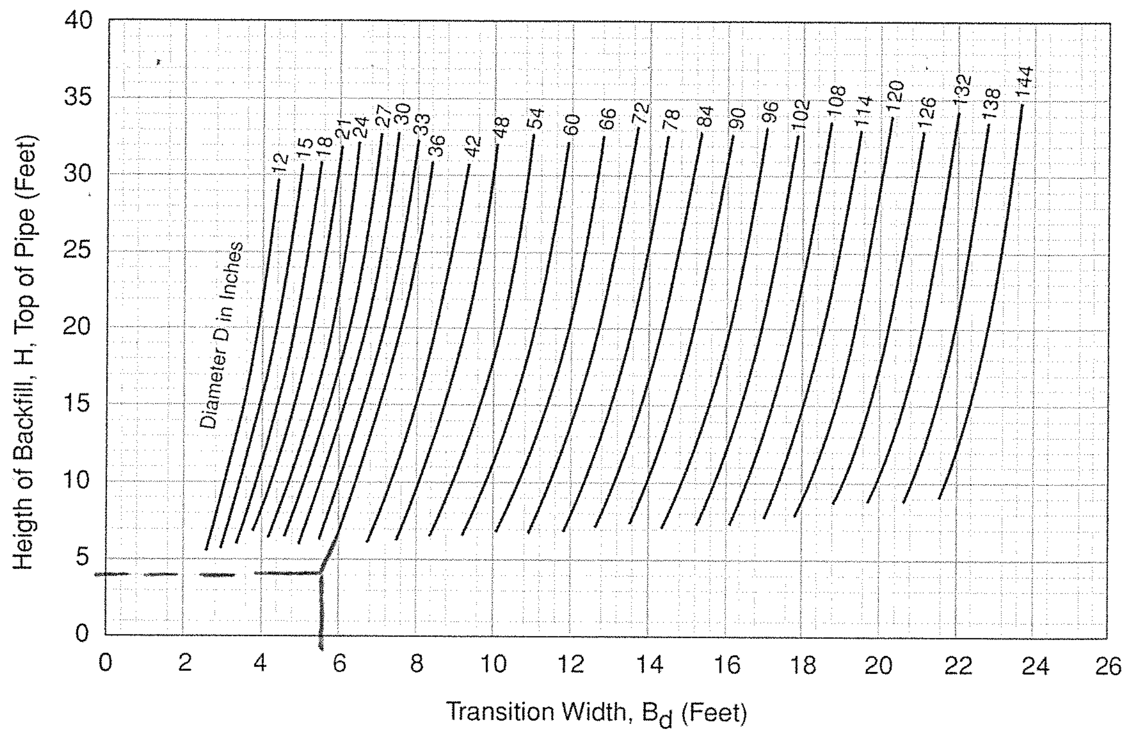
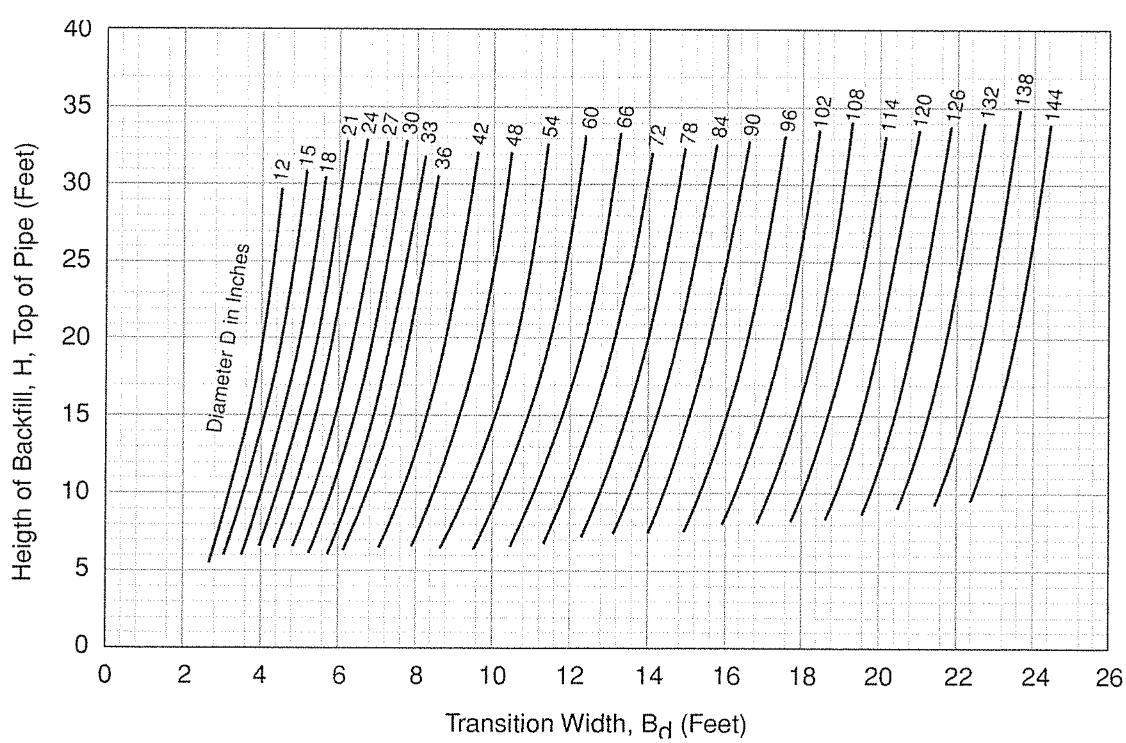


Figure 6 Transition Widths for Type 2 and 3 Installations Using Sand and Gravel Backfill Material



MJ 100 TRAVELIFT® CRANE SPECIFICATIONS

•INDUSTRIAL APPLICATION•

MI-JACK PRODUCTS
3111 West 167th Street, Hazel Crest, Illinois 60429
Phone (708) 596-5200 • Fax (708) 225-2312
www.mi-jack.com
ISO 9001-2000 CERTIFIED

CAPACITY....................................200,000 LBS (90,719KG)

STANDARD EQUIPMENT
Enclosed cab 16' 4" (4.98 m) eye level • Tinted glass • Windshield wiper • Adjustable bucket seat • Operating switches and gauges • Noise suppression • 4 Red strobe lights • 4 Motion alarms • Cummins Diesel QSB6.7 engine with grid heater • (2) dual wheel rear wheel drive • Rear wheel steer • Vivid yellow paint w/black yokes and wheels • Single trolley • Chain traverse • Programmable Electronic Control System (ECS) • Electronic, stepless, infinitely variable controls for hoist, traverse and drive • 18.00 x 25 Bias Ply tires • Wheel guards • Air intake pre-cleaner • 8 Lights (4 work, 4 drive)

ENGINE
Make and Model........................Tier 3 Cummins Diesel QSB6.7
Fuel..No. 2 Diesel
No. of Cylinders..6
Fuel Supply......................................Bosch CP3.3 Electronic
Air Cleaner..Dry Type
Oil Filter...Renewable Cartridge
Cooling System..................................Pressurized Radiator
Horsepower
Gross @ Flywheel..................................260 hp @ 2200 rpm
194 kW @ 2200 rpm
Torque, maximum @ Flywheel................728 lbs. ft. @ 1500 rpm
987 Nm @ 1500 rpm

ELECTRICAL
Voltage.........24 Volt Alternator............70 Amps
Batteries (2)......................1000 CCA @ 0°F (-18°C) for 30 sec.

MAIN HYDRAULICS
Hoist & Traverse Pump..........2 Piston-type load sensing variable displacement
Hoist & Traverse Control....................Four spool sectional valve
Drive Pump...........Piston type over center variable displacement

DRIVE SYSTEM
Hydrostatic on rear wheels. Four piston motors (one at each rear wheel) drive planetary gear transmissions with a roller chain to the drive sprocket at the wheels.
Chain..ANSI 180

BRAKING SYSTEM
Service.......................................Automatic hydrostatic braking
Parking..MultiDisc "SAHR[1]"

STEERING
Electrically controlled hydraulic power rear wheel steer with two double acting cylinders.

TRAVERSE SYSTEM
Direct Chain Drive, one located on each top beam. Each traverse is individually controlled and is driven by a hydraulic motor.
Motor..Radial Piston
Brake... MultiDisc "SAHR[1]"
Chain..ANSI 120

HOIST SYSTEM
Hoist drums are directly coupled to a planetary gearbox, one located on each top beam. Each individually controlled. Each hoist is driven by a fixed displacement piston motor integrated with a direct mounted counterbalance valve.
Motor..Axial Piston
Brake.. MultiDisc "SAHR[1]"
Wire Rope................3/4" EEIPS,Class 6x37, Warrington Seale
Reeving...8 parts per top beam
Sheave Pitch Diameter.................................19.38" (492mm)

PERFORMANCE
Traverse Speed (Rated Capacity)
Speed..60 fpm (18.3 m/min)
Slope...4.0%

2 Speed Hoist
Rated Capacity..................................17.0 fpm (5.2 m/min)
Empty[2]...37.0 fpm (11.3 m/min)

Final Drive Ratio		Level Drive Speed at Rated Capacity Speed	Gradeability at Rated Capacity Paved	Gravel
2 Speed Drive				
2.00:1	Empty	4.9 mph (7.9 km/h)	5.6%	3.5%
	Laden	2.75 mph (4.4 km/h)	5.6%	3.5%

SERVICE CAPACITIES	U.S.	METRIC
Fuel Tank	100 gals	379L
Hydraulic System[3]	75 to 100 gals	284 to 379L
Hydraulic Reservoir	51 gals	193L
Cooling System	33 qts	31L
Engine Oil (w/Filter)	17 qts	16L
Pump Drive Transmission	4 qts	3.8L

OPTIONAL EQUIPMENT
Dual/split trolley (specify spacing) • Top beam widths • Column heights • Side beam lengths • Inward facing cab • Raised operator cab • Ladder safety device(s) • Transverse steer • Cab heater • Engine block heater • Remote control • Open operator station under side beam • Drive camera/monitor kit • Maintenance ladders and platforms for hoists • Air conditioner • AC light package • Power on Demand

ACCESSORIES
Spare tire and wheel • American tool kit (recommended for export) • Filter kits (Hydraulic/engine kits available) • Export preparation • Magnet package[4] • Spreader beams • Spare parts kit

1-"SAHR" spring applied hydraulic release; Automatic Actuation; No mechanical adjustment
2-Contact factory for speeds at loads other than empty.
3-System capacity varies depending on height and width of unit.
4-Consult factory when adding magnet package.

10/22/07

NOTE: MI-JACK PRODUCTS reserves the right to change specifications without notice and without incurring any obligation relating to such a change.

MJ100 TRAVELIFT® CRANE

ESTIMATED SHIPPING WEIGHTS:

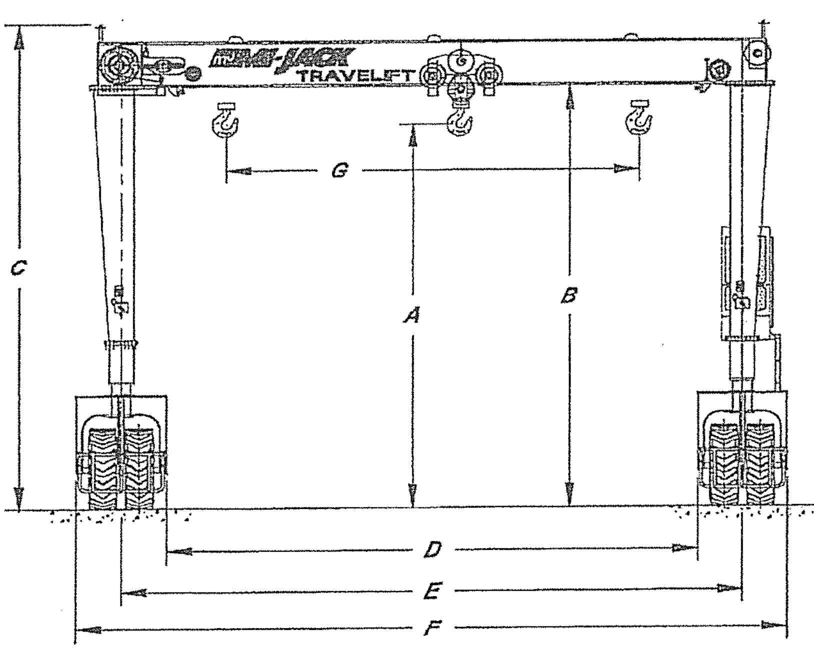
		ICW				
		40' 0"	50' 0"	60' 0"		
HK HT	25' 0"	137,417 lbs 62,331 kg	144,170 lbs 65,394 kg	150,970 lbs 68,479 kg	24' 0"	WB
HK HT	30' 0"	145,225 lbs 65,873 kg	151,978 lbs 68,936 kg	158,778 lbs 72,020 kg	24' 0"	WB
HK HT	40' 0"	149,429 lbs 67,780 kg	156,182 lbs 70,843 kg	162,982 lbs 73,927 kg	24' 0"	WB

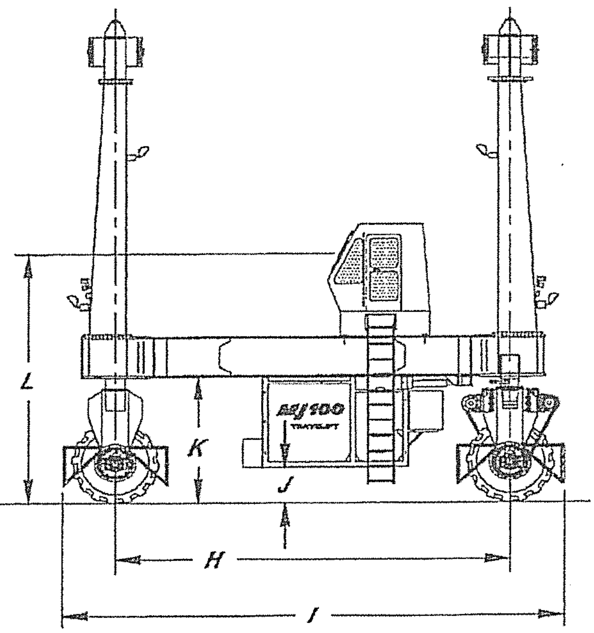
1.) Add 2,950 lbs for 30' WB
2.) Add 7,860 lbs for 40' WB

GROUND BEARING PRESSURE:

		ICW				
		40' 0"	50' 0"	60' 0"		
HK HT	25' 0" MINSPAN	119 psi 820 kPa	122 psi 841 kPa	124 psi 855 kPa	24' 0"	WB
HK HT	30' 0" MINSPAN	120 psi 827 kPa	122 psi 841 kPa	124 psi 855 kPa	24' 0"	WB
HK HT	40' 0" MINSPAN	121 psi 834 kPa	123 psi 848 kPa	126 psi 869 kPa	24' 0"	WB

DIMENSIONAL DATA





Any combination of widths, heights, and lengths shown are available except as noted in charts. Dimensional variations may occur based upon optional equipment characteristics. All crane dimensions are capable of full capacity. Consult factory for optional dimension and ratings.

A HEIGHT TO HK. THROAT(1)	B HEIGHT TO BTTM OF TP BM	C OVERALL HEIGHT	D I.C.W.(2)	E TREAD WIDTH	F OVERALL WIDTH AT GROUND(3)(4)	G TROLLEY TRAVEL(5)
25'- 0" (7.62 m)	27'- 9" (8.46 m)	32'- 0" (9.57 m)	40'- 0" (12.19 m)	45'- 8" (13.92 m)	51'- 5" (15.67 m)	34'- 1" (10.39 m)
30'- 0" (9.14 m)	32'- 9" (9.98 m)	37'- 0" (11.28 m)	50'- 0" (15.24 m)	55'- 8" (16.97 m)	61'- 5" (18.72 m)	44'- 1" (13.44 m)
†40'- 0" (12.19 m)	42'- 9" (13.03 m)	47'- 0" (14.33 m)	60'- 0" (18.29 m)	65'- 8" (20.02 m)	71'- 5" (21.77 m)	54'- 1" (16.48 m)

H WHEELBASE & HK. CENTERS	I OVERALL LENGTH	J GROUND TO ENG. FRAME	K GROUND TO SIDE BEAM	L OPERATOR EYE LEVEL	
24'- 0" (7.32 m)	30'-5" (9.27 m)	1'-6" (0.46 m)	8'- 2" (2.49 m)	Inward facing cab	16'- 4" (4.98 m)
30'- 0" (9.14 m)	36'-5" (11.10 m)			Std. forward cab	16'- 4" (4.98 m)
40'- 0" (12.19 m)	46'-5" (14.15 m)			High inward facing cab	18'- 2" (5.54 m)

(1) Add 3" (0.08 m) for dual trolley option.
(2) Measured from face of yoke.
Subtract 2" (0.05 m) for hardware
(3) For unit with raised cab options add 15" (0.38 m) to left side clearance.
(4) Subtract 6" (0.15 m) for overall width at top.
(5) Single trolley dimensions shown.
For dual trolley, subtract (trolley spacing + 6" (0.15 m))
† Tie bar options

NOTE: All heights above ground include 2" tire deflection for an unloaded crane. Up to 3" additional should be deducted for tire deflection at rated load. Inside, outside and height dimensions are nominal and may vary due to manufacturing standards and structural deflection. 1022/07

NOTE: MI-JACK PRODUCTS reserves the right to change specifications without notice and without incurring any obligation relating to such a change.

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Storm Drain Calculations
Storm Drain Buoyancy Calculations

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OBJECTIVE:

Determine the pipe buoyancy for the 36 inch RCP using light weight concrete being installed as part of the Elm Avenue Storm Drain Relocation project on the AWI property.

GIVENS AND ASSUMPTIONS:

American Concrete Pipe Association (ACPA) Design Manual

The pipe will be installed using Type 1 Bedding. Type 1 Bedding assumes the use of relatively high quality materials and high compaction effort resulting in the need for a lower strength pipe. (ACPA)

The pipe diameter is 36-inch, Wall B.

Assume the pipe is made of light weight concrete (density 100 lb/ft3)

The pipe is to be installed with 4 feet of backfill above the pipe crown.

The soil that the pipe is installed in is assumed to have a surface dry density of 115 pounds per cubic foot and a specific gravity of 2.65.

PROCEDURE:

Find if the pipe would float under conditions of complete backfill, determine the procedures necessary to prevent floatation and what height of backfill is necessary to prevent floatation.

1. **Weight of Pipe**

   Assume the concrete density is 100 lbs/ft^3

$$A = \pi(r_o^2) - \pi(r_i^2)$$

Where:

A= Area if concrete (ft^2)

r_o= Outside radius of pipe (ft)

r_i= Inside radius of pipe (ft)

$$WP = \rho_c \times A$$

Where:

WP= Weight of Pipe

A= Area if concrete (ft^2)

ρ_c= Density of concrete (lb/ft^3)

Solved:

$$A = \pi\left(\frac{22}{12}\right)^2 - \pi\left(\frac{18}{12}\right)^2 = 3.49 \quad ft^2$$

$$WP = 100\ lb/ft^3 \times 3.49\ ft^2 = 349\ lb/ft$$

WP = +349 pounds per linear foot (downward force)

2. Weight of Displaced Water

Based on Table 4[1]: WW = -660 pounds per linear foot (upward force)

WP + WW= +349 + (-660) = -311 pounds per linear foot (upward force)

3. Total Weight of Backfill

Because the resultant force of the pipe and displaced water is upward, the total weight of the backfill must be calculated using the weight of the inundated backfill and weight of the dry backfill. Utilize the following steps:

$$w_I = \rho\left(1 - \frac{1}{\gamma}\right)$$

Where:

w_I = Average Unit Weight of Inundated Backfill (lb/ft^3)

ρ = Surface Dry Density of Soil/Sand (lb/ft^3) = 115 lb/ft^3

γ = Specific Gravity Soil = 2.65

$$W_I = w_I\left(0.1073 B_C{}^2 + H_1 B_c\right)$$

Where:

W_I = Weight of inundated backfill directly over pipe (lb/f)

w_I = Average Unit Weight of Inundated Backfill (lb/ft^3)

B_c = Outside pipe diameter $(ft) = 3.67\ ft$

H_I = Depth of inundated backfill above top of pipe (ft)= $4\ ft$

$$W_D = \rho\left(H - H_1\right)B_c$$

Where:

W_D = Weight of dry backfull directly over pipe (lb/f)

ρ = Surface Dry Density of Soil/Sand (lb/ft^3) = 115 (lb/ft^3)

B_c = Outside pipe diameter $(ft) = 3.67\ ft$

H_I = Depth of inundated backfill above top of pipe (ft)= 0 ft

In the inundated condition the sum of the resultant upward force (from step 2) -311 and the resultant downward force (from step 4) with Factor of Safety of 1.25 becomes +923.74 lb. This condition produces a resultant downward force of +612.75 pounds per linear foot of pipe (923.74 lb - 311 lb = 612.74 lb).

In the dry condition the sum of the resultant upward force (from step 2) -311 and the resultant downward force (from step 4) with Factor of Safety of 1.25 becomes +1350.56 lb. This condition produces a resultant downward force of +1039.56 pounds per linear foot of pipe (1350.56 lb - 311 lb = 1039.56 lb).

CONCLUSIONS:

Since the result force on the pipe is a downward force of +1039.56 in the dry condition and in the inundated condition the downward force is +612.75 pounds per linear foot of pipe based on a Factor of Safety of 1.25, the pipe will not float once backfill placement is completed to a height of 4 feet above the pipe crown.

$$W_B = W_I + W_D$$

Where:

$W_B =$ Total weight of backfill directly over pipe (lb/f^3)

$W_I =$ Weight of inundated backfill directly over pipe (lb/f)

$W_D =$ Weight of dry backfill directly over pipe (lb/f)

Solved:

$$w_I = 115\left(1 - \frac{1}{2.65}\right) = 71.60\ lb/ft^3$$

$$W_I = 71.60\left[0.1073(3.67)^2 + 4(3.67)\right] = 1154.68\ \ lb/ft$$

Since the groundwater table was assumed to be at the ground surface, there would be no additional downward force. $W_D = 0$

$$W_B = 1154.68 + 0 = 1154.68\ lb/ft$$

For the condition where the backfill material over pipe is dry $W_D =$ 1688.20 lb/ft.

4. **Application of Factor of Safety**
Since no precise information is available on the density and specific gravity of the soil, a Factor of Safety of 1.25 will be used to reduce the assumed total weight of the backfill.

$$\frac{W_B}{F.S.}$$

Solved:

For Saturate Conditions:

$$\frac{W_B}{F.S.} = \frac{1154.68}{1.25} = 923.74\ \ lb\ \ (downward\ \ force)$$

For Dry Conditions:

$$\frac{W_B}{F.S.} = \frac{1688.20}{1.25} = 1350.56\ \ lb\ \ (downward\ \ force)$$

Appendix E

Hydraulic Calculations

- **Hydraulic Calculations - Pre-Development**
- **Hydraulic Calculations - Post-Development**
- **Post-Development Model Schematic**
- **Hydraulic Summary Table**

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Hydraulic Calculations
Pre-Development

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Project	Atlantic Wood Industries Superfund Site	Project No.	1453011
Subject	Pre-Development Hydraulic Model	Sheet No.	1 of 1
		Drawing No.	
Computed by JLL	Date 6/4/12	Checked by GAT	Date 6/26/12

OBJECTIVE:

Determine the limits of flooding during the 10 year 24 hour design storm event in the pre-development condition for the Elm Avenue Storm Drain.

PROCEDURE, ASSUMPTIONS, & CALCULATIONS:

Use U.S. Army Corps of Engineers Hydrologic Engineering Center Riverine Analysis System (HEC-RAS) to perform a hydraulic analysis of the pre-development conditions of the Elm Avenue Storm Drain for the 10 year 24 hour design storm along with the three tidal elevations scenarios; Mean High Water (Elev. 1.17 NAVD88), Mean Sea Level (Elev. -0.25 NAVD88) and Mean Low Water (Elev. -1.69 NAVD88).

- All models assumed storm drain was surcharged at time of peak runoff. Peak runoff for all sub basins routed overland using model.
- Models include boundary condition at the downstream cross section equal to the tidal elevations of the model.
- Flows equal to sub basin peak runoff were added at cross sections downstream of sub basin discharge point to existing storm drain.

U.S. Army Corps of Engineers Hydrologic Engineering Center – Riverine Analysis System (HEC-RAS)

Separate hydraulic models were created for the three tidal scenarios to evaluate the 10 year 24 hour storm event. Existing and proposed sub-basins were developed using TR-55 method for determining curve numbers and time of concentration.

The tidal elevations for each scenario were input as a know water surface elevation at the downstream cross section.

Solved:

The hydraulic model computed limits of flooding for the 10 year 24 hour storm for each tidal scenario.

CONCLUSIONS:

The limits of flooding determined by the hydraulic models for the three tidal scenarios are nearly identical with only minor differences in the area where the stormwater runoff discharges into the Southern Branch of the Elizabeth River. The limits of inundation are shown on the attached figures. The volume of runoff from the project watershed is insignificant compared to the volume of the river and therefore the tidal elevations do not affect the limits of flooding. The hydraulic models confirm the flooding described by adjacent property owners and the existing storm drains limited capacity and the inability to convey the 10 year 24 hour storm as presently configured.

The output for tidal elevation scenarios Mean Low Water, Mean Sea Level and Mean High Water and Area of Inundation Figures 1, 2 and 3 are included in this Appendix.

```
                         HEC-RAS Version 4.1.0 Jan 2010
                          U.S. Army Corps of Engineers
                         Hydrologic Engineering Center
                               609 Second Street
                               Davis, California

            X     X  XXXXXX   XXXX        XXXX     XX    XXXX
            X     X  X       X    X       X   X   X  X  X
            X     X  X       X            X   X  X    X X
            XXXXXXX  XXXX    X      XXX   XXXX   XXXXXX  XXXX
            X     X  X       X            X X    X    X      X
            X     X  X       X    X       X  X   X    X      X
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Run Date and Time: 6/19/2012 3:40:16 PM

Project in English units

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PLAN DATA

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           Flow File     : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.f04

Plan Summary Information:
Number of:  Cross Sections =   15    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Conveyance
    Computational Flow Regime:     Subcritical Flow
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Flow Data (cfs)

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* River           Reach           RS      *        10 year *
* Elm_Ave         1               2101    *              9 *
* Elm_Ave         1               1260    *             33 *
* Elm_Ave         1               768     *            100 *
```

****

Boundary Conditions

****

River	Reach	Profile	Upstream	Downstream
Elm_Ave	1	10 year	Known WS = 1.17	

****

****

SUMMARY OF MANNING'S N VALUES

River:Elm_Ave

Reach	River Sta.	n1	n2	n3	n4	n5	n6
1	2101	.012					
1	1997	.03	.012	.03	.012	.03	
1	1890	.03	.012	.03			
1	1770	.03	.012	.03			
1	1671	.012	.03	.012	.03		
1	1546	.012	.03	.012	.03		
1	1423	.012	.03	.012	.03		
1	1260	.012	.03	.12	.03		
1	1118	.012	.03	.012	.03		
1	995	.012	.03	.012	.03	.012	.03
1	883	.012	.03	.012	.03		
1	768	.012	.03	.012			
1	707	.03	.035	.015	.035	.03	
1	531	.03	.015	.03	.015	.03	
1	110	.035	.05	.03	.015	.03	

****

SUMMARY OF REACH LENGTHS

River: Elm_Ave

Reach	River Sta.	Left	Channel	Right
1	2101	103.96	103.57	102.04
1	1997	106.06	106.67	109.68
1	1890	120.81	120.31	117.13
1	1770	101.97	98.65	97.53
1	1671	127.87	125.5	130.26
1	1546	125.55	122.82	119.81
1	1423	165.27	163.34	162.38
1	1260	143.68	141.76	144.67
1	1118	122.75	123.21	121.97
1	995	109.54	111.4	108.53
1	883	104.72	115.02	107.96
1	768	64.5	60.9	83.63
1	707	176.82	176.25	178.43
1	531	450.51	421.47	390.96
1	110	131.05	109.72	89.89

Profile Output Table - Standard Table 2

Reach	River Sta	Profile	E.G. Elev (ft)	W.S. Elev (ft)	Vel Head (ft)	Frctn Loss (ft)	C & E Loss (ft)	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Top Width (ft)
1	2101	10 year	8.17	8.13	0.03	0.00	0.01		0.04	8.96	95.55
1	1997	10 year	7.76	7.76	0.00	0.00	0.00		1.60	7.40	224.13
1	1890	10 year	7.76	7.76	0.00	0.00	0.00		2.27	6.73	237.40
1	1770	10 year	7.76	7.76	0.00	0.00	0.00		2.94	6.06	266.91
1	1671	10 year	7.75	7.73	0.02	0.38	0.00	0.00	8.95	0.04	61.55
1	1546	10 year	7.38	7.31	0.07	0.10	0.02		3.64	5.36	38.36
1	1423	10 year	6.81	6.80	0.01	0.22	0.00		9.00		44.72
1	1260	10 year	6.58	6.56	0.02	0.47	0.00		5.82	27.18	105.59
1	1118	10 year	6.11	6.08	0.03	0.83	0.01			33.00	92.45
1	995	10 year	5.27	5.17	0.10	0.43	0.02		28.66	4.34	67.18
1	883	10 year	4.77	4.75	0.02	0.30	0.01		33.00		78.67
1	768	10 year	4.45	4.34	0.11	0.27	0.04	83.89	16.11		165.37
1	707	10 year	3.24	2.78	0.47	0.89	0.08		100.00		19.78
1	531	10 year	1.82	1.62	0.21	0.00	0.06		100.00		65.94
1	110	10 year	1.17	1.17	0.00				100.00		432.36

FILE PATH: (LOVETON) L:\NON DOD\AWI CAD\PHASE 2\HYD\EASDR\EASDR EX FLOOD.DWG [10 YR - MHW] 6/19/12

EA® EA ENGINEERING, SCIENCE, AND TECHNOLOGY

REMEDIAL DESIGN
ELM AVENUE STORM DRAIN RELOCATION
ATLANTIC WOOD INDUSTRIES SUPERFUND SITE
PORTSMOUTH, VIRGINIA

PRE-DEVELOPMENT DRAINAGE CONDITIONS
AREA OF INUNDATION
10 YEAR 24 HOUR STORM AT MEAN HIGH WATER

DESIGNED BY	DRAWN BY	DATE	PROJECT NO.
JLL	JLL	JUNE 2012	14530.11
CHECKED BY	PROJECT MGR.	SCALE	FIGURE
GAT	PAP	1"=150'	1

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                    HEC-RAS Version 4.1.0 Jan 2010
                     U.S. Army Corps of Engineers
                    Hydrologic Engineering Center
                          609 Second Street
                          Davis, California

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         XXXXXXX  XXXX     X       XXX  XXXX     XXXXXX    XXXX
         X     X  X        X            X  X     X    X        X
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Run Date and Time: 6/19/2012 3:46:17 PM

Project in English units

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PLAN DATA

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Geometry Title: Existing
Geometry File : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.g01

Flow Title   : MLW boundary condition
Flow File   : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.f03

Plan Summary Information:

Number of:				
	Cross Sections =	15	Multiple Openings =	0
	Culverts =	0	Inline Structures =	0
	Bridges =	0	Lateral Structures =	0

Computational Information
- Water surface calculation tolerance = 0.01
- Critical depth calculation tolerance = 0.01
- Maximum number of iterations = 20
- Maximum difference tolerance = 0.3
- Flow tolerance factor = 0.001

Computation Options
- Critical depth computed only where necessary
- Conveyance Calculation Method: At breaks in n values only
- Friction Slope Method: Average Conveyance
- Computational Flow Regime: Subcritical Flow

****

FLOW DATA

Flow Title: MLW boundary condition
Flow File : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.f03

Flow Data (cfs)

River	Reach	RS	10 year
Elm_Ave	1	2101	9
Elm_Ave	1	1260	33
Elm_Ave	1	768	100

Boundary Conditions

River	Reach	Profile	Upstream	Downstream
Elm_Ave	1	10 year	Known WS = -1.69	

SUMMARY OF MANNING'S N VALUES

River:Elm_Ave

Reach	River Sta.	n1	n2	n3	n4	n5	n6
1	2101	.012					
1	1997	.03	.012	.03	.012	.03	
1	1890	.03	.012	.03			
1	1770	.03	.012	.03			
1	1671	.012	.03	.012	.03		
1	1546	.012	.03	.012	.03		
1	1423	.012	.03	.012	.03		
1	1260	.012	.03	.12	.03		
1	1118	.012	.03	.012	.03		
1	995	.012	.03	.012	.03	.012	.03
1	883	.012	.03	.012	.03		
1	768	.012	.03	.012			
1	707	.03	.035	.015	.035	.03	
1	531	.03	.015	.03	.015	.03	
1	110	.035	.05	.03	.015	.03	

SUMMARY OF REACH LENGTHS

River: Elm_Ave

Reach	River Sta.	Left	Channel	Right
1	2101	103.96	103.57	102.04
1	1997	106.06	106.67	109.68
1	1890	120.81	120.31	117.13
1	1770	101.97	98.65	97.53
1	1671	127.87	125.5	130.26
1	1546	125.55	122.82	119.81
1	1423	165.27	163.34	162.38
1	1260	143.68	141.76	144.67
1	1118	122.75	123.21	121.97
1	995	109.54	111.4	108.53
1	883	104.72	115.02	107.96
1	768	64.5	60.9	83.63
1	707	176.82	176.25	178.43
1	531	450.51	421.47	390.96
1	110	131.05	109.72	89.89

Profile Output Table - Standard Table 2

Reach	River Sta	Profile	E.G. Elev (ft)	W.S. Elev (ft)	Vel Head (ft)	Frctn Loss (ft)	C & E Loss (ft)	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Top Width (ft)
1	2101	10 year	8.17	8.13	0.03	0.00	0.01		0.04	8.96	95.55
1	1997	10 year	7.76	7.76	0.00	0.00	0.00		1.60	7.40	224.13
1	1890	10 year	7.76	7.76	0.00	0.00	0.00		2.27	6.73	237.40
1	1770	10 year	7.76	7.76	0.00	0.00	0.00		2.94	6.06	266.91
1	1671	10 year	7.75	7.73	0.02	0.38	0.00	0.00	8.95	0.04	61.55
1	1546	10 year	7.38	7.31	0.07	0.10	0.02		3.64	5.36	38.36
1	1423	10 year	6.81	6.80	0.01	0.22	0.00		9.00		44.72
1	1260	10 year	6.58	6.56	0.02	0.47	0.00		5.82	27.18	105.59
1	1118	10 year	6.11	6.08	0.03	0.83	0.01			33.00	92.45
1	995	10 year	5.27	5.17	0.10	0.43	0.02		28.66	4.34	67.18
1	883	10 year	4.77	4.75	0.02	0.30	0.01		33.00		78.67
1	768	10 year	4.45	4.34	0.11	0.27	0.04	83.89	16.11		165.37
1	707	10 year	3.24	2.78	0.47	0.89	0.08		100.00		19.78
1	531	10 year	1.82	1.62	0.21	2.11	0.03		100.00		65.94
1	110	10 year	-0.69	-0.79	0.10				100.00		205.90

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LEGEND

10	EXISTING INDEX CONTOUR
9	EXISTING INTERMEDIATE CONTOUR
	EXISTING BUILDING
	EXISTING WETLAND
	EXISTING DITCH
	EXISTING GRAVEL
	EXISTING PAVEMENT
	EXISTING STORMDRAIN
	EXISTING PROPERTY LINE
	AREA OF INUNDATION

FILE PATH: (LOVETON) L:\NON DOD\AWI CAD\PHASE 2\HYD\EASDR\EASDR EX FLOOD.DWG [10 YR - MSL] 6/19/12

EA® EA ENGINEERING, SCIENCE, AND TECHNOLOGY

REMEDIAL DESIGN
ELM AVENUE STORM DRAIN RELOCATION
ATLANTIC WOOD INDUSTRIES SUPERFUND SITE
PORTSMOUTH, VIRGINIA

PRE-DEVELOPMENT DRAINAGE CONDITIONS
AREA OF INUNDATION
10 YEAR 24 HOUR STORM AT MEAN SEA LEVEL

DESIGNED BY	DRAWN BY	DATE	PROJECT NO.
JLL	JLL	JUNE 2012	14530.11
CHECKED BY	PROJECT MGR.	SCALE	FIGURE
GAT	PAP	1"=150'	1

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HEC-RAS Version 4.1.0 Jan 2010
U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California

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X     X  XXXXXX   XXXX        XXXX     XX    XXXX
X     X  X       X    X       X   X   X  X  X
X     X  X       X            X   X  X    X X
XXXXXXX  XXXX    X      XXX   XXXX   XXXXXX  XXXX
X     X  X       X            X  X   X    X      X
X     X  X       X    X       X   X  X    X      X
X     X  XXXXXX   XXXX        X    X X    X XXXXX
```

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PROJECT DATA
Project Title: Elm_Ave_Drainage
Project File : EASDR_Pre.prj
Run Date and Time: 6/19/2012 3:47:12 PM

Project in English units

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PLAN DATA

Plan Title: MSL boundary condition
Plan File : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.p06

Geometry Title: Existing
Geometry File : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.g01

Flow Title : MSL boundary condition
Flow File : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.f05

```
Plan Summary Information:
Number of:  Cross Sections =   15    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Conveyance
    Computational Flow Regime:     Subcritical Flow
```

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FLOW DATA

Flow Title: MSL boundary condition
Flow File : l:\Non Dod\AWI CAD\Phase 2\HYD\EASDR\EASDR_Pre.f05

Flow Data (cfs)

River	Reach	RS	10yr
Elm_Ave	1	2101	9
Elm_Ave	1	1260	33
Elm_Ave	1	768	100

Boundary Conditions

River	Reach	Profile	Upstream	Downstream
Elm_Ave	1	10yr	Known WS = -0.25	

SUMMARY OF MANNING'S N VALUES

River:Elm_Ave

Reach	River Sta.	n1	n2	n3	n4	n5	n6
1	2101	.012					
1	1997	.03	.012	.03	.012	.03	
1	1890	.03	.012	.03			
1	1770	.03	.012	.03			
1	1671	.012	.03	.012	.03		
1	1546	.012	.03	.012	.03		
1	1423	.012	.03	.012	.03		
1	1260	.012	.03	.12	.03		
1	1118	.012	.03	.012	.03		
1	995	.012	.03	.012	.03	.012	.03
1	883	.012	.03	.012	.03		
1	768	.012	.03	.012			
1	707	.03	.035	.015	.035	.03	
1	531	.03	.015	.03	.015	.03	
1	110	.035	.05	.03	.015	.03	

SUMMARY OF REACH LENGTHS

River: Elm_Ave

Reach	River Sta.	Left	Channel	Right
1	2101	103.96	103.57	102.04
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1	1770	101.97	98.65	97.53
1	1671	127.87	125.5	130.26
1	1546	125.55	122.82	119.81
1	1423	165.27	163.34	162.38
1	1260	143.68	141.76	144.67
1	1118	122.75	123.21	121.97
1	995	109.54	111.4	108.53
1	883	104.72	115.02	107.96
1	768	64.5	60.9	83.63
1	707	176.82	176.25	178.43
1	531	450.51	421.47	390.96
1	110	131.05	109.72	89.89

Profile Output Table - Standard Table 2

Reach	River Sta	Profile	E.G. Elev (ft)	W.S. Elev (ft)	Vel Head (ft)	Frctn Loss (ft)	C & E Loss (ft)	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Top Width (ft)
1	2101	10yr	8.17	8.13	0.03	0.00	0.01		0.04	8.96	95.55
1	1997	10yr	7.76	7.76	0.00	0.00	0.00		1.60	7.40	224.13
1	1890	10yr	7.76	7.76	0.00	0.00	0.00		2.27	6.73	237.40
1	1770	10yr	7.76	7.76	0.00	0.00	0.00		2.94	6.06	266.91
1	1671	10yr	7.75	7.73	0.02	0.38	0.00	0.00	8.95	0.04	61.55
1	1546	10yr	7.38	7.31	0.07	0.10	0.02		3.64	5.36	38.36
1	1423	10yr	6.81	6.80	0.01	0.22	0.00		9.00		44.72
1	1260	10yr	6.58	6.56	0.02	0.47	0.00		5.82	27.18	105.59
1	1118	10yr	6.11	6.08	0.03	0.83	0.01			33.00	92.45
1	995	10yr	5.27	5.17	0.10	0.43	0.02		28.66	4.34	67.18
1	883	10yr	4.77	4.75	0.02	0.30	0.01		33.00		78.67
1	768	10yr	4.45	4.34	0.11	0.27	0.04	83.89	16.11		165.37
1	707	10yr	3.24	2.78	0.47	0.89	0.08		100.00		19.78
1	531	10yr	1.82	1.62	0.21	0.09	0.06		100.00		65.94
1	110	10yr	-0.24	-0.25	0.01				100.00		291.33

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EA® EA ENGINEERING, SCIENCE, AND TECHNOLOGY	REMEDIAL DESIGN ELM AVENUE STORM DRAIN RELOCATION ATLANTIC WOOD INDUSTRIES SUPERFUND SITE PORTSMOUTH, VIRGINIA	PRE-DEVELOPMENT DRAINAGE CONDITIONS AREA OF INUNDATION 10 YEAR 24 HOUR STORM AT MEAN LOW WATER	DESIGNED BY JLL	DRAWN BY JLL	DATE JUNE 2012
			CHECKED BY GAT	PROJECT MGR. PAP	SCALE 1"=150'

PROJECT NO. 14530.11

FIGURE 1

FILE PATH: (LOVETON) L:\NON DOD\AWI CAD\PHASE 2\HYD\EASDR\EASDR EX FLOOD.DWG [10 YR - MLW] 6/19/12

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Hydraulic Calculations
Post-Development

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Project	Atlantic Wood Industries Superfund Site	Project No.	1453011
Subject	Post-Development Hydraulic Model	Sheet No.	1 of 3
		Drawing No.	
Computed by JLL	Date 6/4/12	Checked by GAT	Date 6/26/12

OBJECTIVE:

Determine the affects of tidal water surface elevations on the Elm Avenue Storm Drain.

PROCEDURE, ASSUMPTIONS, LIMITATIONS & CALCULATIONS:

Use Autodesk Storm and Sanitary Analysis 2012 to perform a hydrologic and hydraulic analysis of the post-development conditions of the Elm Avenue Storm Drain Relocation project for the 100 year 24 hour, 50 year 24 hour, 25 year 24 hour, 10 year 24 hour, 5 year 24 hour and 2 year 24 hour. Individual models were created for three tidal elevations scenarios; Mean High Water (Elev. 1.17 NAVD88), Mean Sea Level (Elev. -0.25 NAVD88) and Mean Low Water (Elev. -1.69 NAVD88).

NRCS Technical Release 55 "Urban Hydrology for Small Watersheds"

Autodesk Storm and Sanitary Analysis 2012

- Hydrologic curve numbers and time of concentration based on Remedial Action final conditions.
- All tidal scenarios included a tide valve at the outlet of the model.
- All tidal scenarios included an initial water elevation in the pipes equal to the tidal elevations of the model.
- Tidal elevations in all scenarios were fixed during the entire 24 hour simulation.
- The model uses hydrodynamic routing method.
- Generally in coastal regions large storm events (25 year, 50 year, and 100 year) are associated with hurricanes, tropical storms, etc. generating large rainfall amounts creating large storm surges in the river along with large stormwater runoff from upland areas. Theses storm surges in the river create significant flooding of the low ground elevations on the AWI and adjacent properties by inundation from the rising water surface elevation. More frequent storm events (2 year, 5 year, and 10 year) can result from locally occurring weather systems over smaller areas and therefore not cause significant rises in river elevations.
- The intent of the Elm Avenue Storm Drain Relocation project is to create conveyance for existing flows and capacity for flows generated by the creation of the new land; the project is not intended to provide flood control. The models do not take into account storm surge in the river due to hurricanes, tropical storms, and other severe weather systems. **During the severe weather systems the AWI and adjacent properties will experience flooding and inundation regardless of the Elm Avenue storm drain and therefore have not been considered in this design.**

Separate hydraulic models were created for the three tidal scenarios to evaluate all storm events listed above. Existing and proposed sub basins were developed using TR-55 method for determining curve numbers and time of concentration. The proposed sub basins included the newly-created land that will be generated as a result of the placement and capping of the dredged material from the river. Flows from existing sub basins near Burton's Point Road were routed through the existing storm sewer in Elm Avenue which connects to the proposed Elm Avenue relocated storm drain. Because the existing storm drain is under-sized, much of the runoff is conveyed within the roadway section of Elm Avenue consisting of using an overland conduit in the model. The flows are captured at the next downstream junction and conveyed through the pipe or allowed to overtop the junction and continue flowing overland until a junction with capacity is reached. The model schematic for all models is included as Figure 4 – Post-Development Conditions Model Schematic.

For junctions in sump conditions, a stage storage relationship was developed for the surface above the rim and overtopping elevations are defined at which time flows are conveyed via the overland conduit. The model allows ponding at junctions for runoff that exceed the rim elevation of the junction but does not achieve the overtopping elevation and therefore no runoff is lost from the model.

The tidal elevations for each scenario were input at the outlet node of the model as a fixed elevation for the discharge point. The presence of the tide valve in the model restricts backflow from the river into the system; however, between rain events the storm drain will only empty to an elevation equal to water surface elevation of the river. To produce the condition of standing water in the pipe at the beginning of the analysis, an initial depth was input at each junction upstream equal to the fixed tidal elevation of the outlet.

Solved:

The hydraulic model computed a maximum hydraulic grade lines for each storm in each scenario. Maximum hydraulic grade elevations at each storm drain junction for the considered storm events analyzed in each scenario are shown in the attached table. The following attachments include Figure 4 – Post-Development Conditions Model Schematic, the model output for all three tidal elevation scenarios and storm drain profiles showing typical standing water in pipe for each tidal scenario.

CONCLUSIONS:

In accordance with the City of Portsmouth requirements the storm drain will convey the 10 year storm peak runoff in all tidal scenarios. In the event of the 10 year, 24 hour design storm affecting the watershed for the Southern Branch of the Elizabeth River the short time of concentration for the project watershed would allow the peak runoff to discharge to the river in advance of the rise in river elevation. However, the AWI and surrounding properties will still incur flooding from inundation when the river elevation rises to the 10 year, 24 hour design storm elevation of 5.5.

As stated in the assumptions; the large storm events (25 year, 50 year and 100 year) are associated with severe weather systems; and therefore, are not considered in the designs. However models were run for all storm events to illustrate the maximum discharge for the system during the modeled tidal scenarios. Maximum hydraulic grade and discharge for all storm events are shown in Table 1 – Summary of Hydraulic Modeling. Hydraulic model output for the 10 year, 24 hour design storm is included in this appendix, output for the other storm events have not been included.

Based on the hydraulic model scenarios for the design storms listed, the proposed storm drain functions effectively for the 2, 5, 10, 25 and 50 year storm events for the Mean Low Water tidal scenario. During the 100 year storm event for the Mean Low Water tidal scenario the maximum hydraulic grade elevation exceeds the rim elevation at SD Junction Box 4 and the Veneer Road Curb Inlet for a short

period.

Similarly, during the 2, 5, 10, and 25 year storm event it functions effectively for the Mean Sea Level tidal scenario, with the maximum hydraulic grade line exceeding the rim elevations for a short period at SD Junction Box 4 and Veneer Road Curb Inlet in the 50 and 100 year events.

For the Mean High Water tidal scenario the storm drain functions effectively in the 2, 5 and 10 year storms. SD Junction Box 4 and Veneer Road Curb Inlet rim elevations are exceeded for a short period in the 25, 50 and 100 year storm events.

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Mean High Water Tide Scenario

Prepared by JL
Checked by GAT

Junction Results

SN	Element ID	Peak Inflow (cfs)	Peak Lateral Inflow (cfs)	Max HGL Elevation Attained (ft)	Max HGL Depth Attained (ft)	Max Surcharge Depth Attained (ft)	Min Freeboard Attained (ft)	Average HGL Elevation Attained (ft)	Average HGL Depth Attained (ft)	Time of Max HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-in)	Total Time Flooded (min)
1	E2	23.38	0.00	5.73	4.60	0.00	0.50	1.50	0.37	0 12:20	0 00:00	0.00	0.00
2	E4	5.34	0.00	6.59	5.16	0.00	0.00	1.84	0.41	0 12:04	0 12:04	0.00	0.00
3	E6	1.50	0.00	7.58	3.09	0.00	0.36	4.61	0.12	0 12:19	0 00:00	0.00	0.00
4	E7	1.50	1.50	7.62	1.63	0.00	0.08	6.14	0.15	0 12:17	0 00:00	0.00	0.00
5	J2	29.58	0.00	3.25	5.02	0.00	0.98	1.22	2.99	0 12:18	0 00:00	0.00	0.00
6	SD1	106.45	0.00	1.44	6.29	0.00	8.06	1.16	6.01	0 12:18	0 00:00	0.00	0.00
7	SD2	98.96	0.00	3.05	5.94	0.00	2.19	1.21	4.10	0 12:20	0 00:00	0.00	0.00
8	SD3	90.12	28.07	3.61	5.52	0.00	1.83	1.22	3.13	0 12:22	0 00:00	0.00	0.00
9	SD4	43.80	12.88	3.81	5.39	0.00	0.66	1.23	2.81	0 12:21	0 00:00	0.00	0.00
10	SWM1	15.97	15.97	1.96	1.96	0.00	2.84	1.17	1.17	0 12:09	0 00:00	0.00	0.00
11	SWM2	14.70	14.70	3.49	4.16	0.00	1.84	1.22	1.89	0 12:16	0 00:00	0.00	0.00
12	SWM3	23.82	14.89	4.48	6.00	0.00	0.00	1.22	2.74	0 00:00	0 00:00	0.00	0.00
13	SWM4	8.27	8.27	4.09	3.28	0.00	2.72	1.25	0.44	0 12:19	0 00:00	0.00	0.00
14	V1	8.26	0.00	3.98	3.28	0.00	0.26	1.24	0.54	0 12:20	0 00:00	0.00	0.00

Mean High Water Tide Scenario

Prepared by JL
Checked by GAT

Channel Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	E5Overland	0.40	0 12:20	7811.17	0.00	50.00	0.19	0.12	0.02	0.00		
2	NavyOverland	0.00	0 00:00	17830.75	0.00	0.00		0.48	0.17	0.00		
3	VermeerOverland	21.36	0 12:18	1807.37	0.01	0.25	22.67	0.73	0.56	0.00		

Mean High Water Tide Scenario

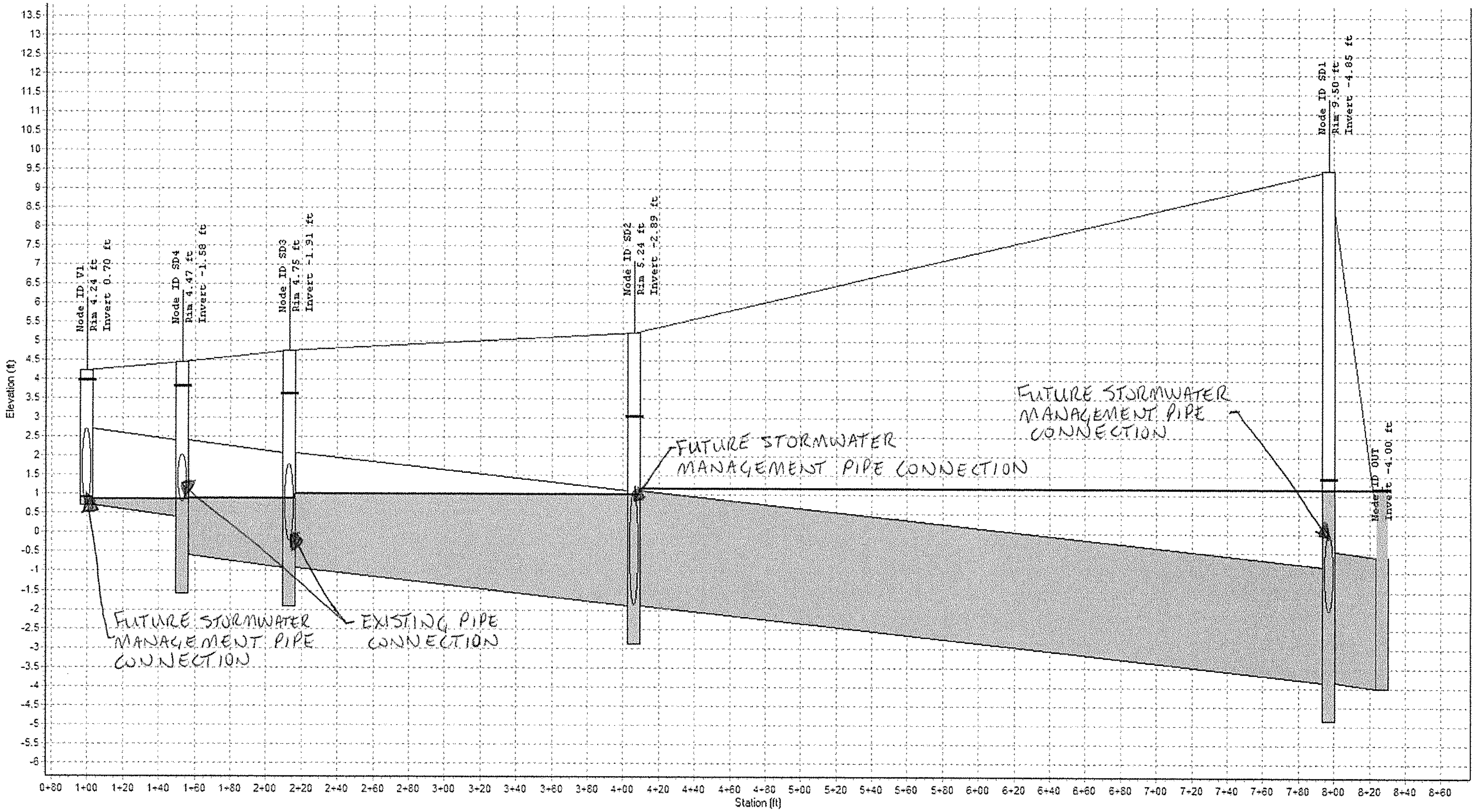
Prepared by JL
Checked by GAT

Pipe Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	E2-E1	23.38	0 12:12	16.43	1.42	7.44	0.67	2.00	1.00	32.00		SURCHARGED
2	E3-E2	23.38	0 12:12	2.85	8.21	7.44	0.17	2.00	1.00	31.00		SURCHARGED
3	E4-E3	5.41	0 12:34	4.57	1.18	1.72	1.40	2.00	1.00	32.00		SURCHARGED
4	E5-E4	5.34	0 12:30	6.88	0.78	3.32	3.46	1.50	1.00	26.00		SURCHARGED
5	E6-E5	1.54	0 12:10	5.39	0.29	1.64	0.98	1.25	1.00	26.00		SURCHARGED
6	E7-E6	1.50	0 12:12	0.83	1.80	3.00	0.24	1.00	1.00	20.00		SURCHARGED
7	J2-SD2	29.57	0 12:12	63.32	0.47	4.18	0.04	3.00	1.00	102.00		SURCHARGED
8	SD1-OUT	106.45	0 12:18	216.82	0.49	3.87	0.13	3.42	1.00	1440.00		SURCHARGED
9	SD2-SD1	98.96	0 12:21	122.78	0.81	4.67	1.40	3.00	1.00	1334.00		SURCHARGED
10	SD3-SD2	80.63	0 12:27	102.98	0.78	5.70	0.56	3.00	1.00	37.00		SURCHARGED
11	SD4-SD3	42.65	0 12:16	80.38	0.53	3.02	0.33	3.00	1.00	31.00		SURCHARGED
12	SWM1-SD1	15.95	0 12:09	46.21	0.35	5.09	0.12	1.98	0.99	0.00		Calculated
13	SWM2-J2	14.70	0 12:12	35.55	0.41	3.11	0.59	2.50	1.00	43.00		SURCHARGED
14	SWM3-J2	23.82	0 00:00	35.55	0.67	5.11	0.08	2.50	1.00	1366.00		SURCHARGED
15	SWM4-V1	8.26	0 12:15	20.56	0.40	2.63	0.06	2.00	1.00	24.00		SURCHARGED
16	V1-SD4	8.25	0 12:16	14.25	0.58	2.63	0.34	2.00	1.00	25.00		SURCHARGED
17	V2-V1	8.19	0 12:48	2.25	3.64	6.75	0.84	1.25	1.00	41.00		SURCHARGED

Elm Avenue Storm Drain Relocation

Typical Standing Water in Pipe at Mean High Water Initial Depth = 1.17 NAVD88



Prepared by JL
Checked by CM

Junction Results

SN	Element ID	Peak Inflow (cfs)	Peak Lateral Inflow (cfs)	Max HGL Elevation Attained (ft)	Max HGL Depth Attained (ft)	Max Surcharge Depth Attained (ft)	Min Freeboard Attained (ft)	Average HGL Elevation Attained (ft)	Average HGL Depth Attained (ft)	Time of Max HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-in)	Total Time Flooded (min)
1	E2	24.44	0.00	5.42	4.29	0.00	0.81	1.65	0.52	0 12:16	0 00:00	0.00	0.00
2	E4	6.86	0.00	6.59	5.16	0.00	0.00	1.88	0.45	0 12:02	0 00:00	0.00	0.00
3	E6	1.50	0.00	7.90	3.41	0.00	0.04	4.64	0.15	0 12:10	0 00:00	0.00	0.00
4	E7	1.50	1.50	7.70	1.71	0.00	0.00	6.15	0.16	0 12:09	0 12:09	0.00	0.00
5	J2	29.47	0.00	1.20	2.97	0.00	3.03	-1.35	0.42	0 12:15	0 00:00	0.00	0.00
6	SD1	124.38	0.00	-1.00	3.85	0.00	10.50	-1.66	3.19	0 12:12	0 00:00	0.00	0.00
7	SD2	113.59	0.00	0.90	3.79	0.00	4.34	-1.44	1.45	0 12:15	0 00:00	0.00	0.00
8	SD3	86.52	28.07	1.41	3.32	0.00	4.03	-0.52	1.39	0 12:17	0 00:00	0.00	0.00
9	SD4	42.89	12.88	1.67	3.25	0.00	2.80	-0.24	1.34	0 12:20	0 00:00	0.00	0.00
10	SWM1	15.97	15.97	1.11	1.11	0.00	3.69	0.16	0.16	0 12:06	0 00:00	0.00	0.00
11	SWM2	14.70	14.70	1.46	2.13	0.00	3.87	-0.43	0.24	0 12:15	0 00:00	0.00	0.00
12	SWM3	14.89	14.89	1.38	2.90	0.00	3.10	-1.21	0.31	0 12:15	0 00:00	0.00	0.00
13	SWM4	8.28	8.28	2.30	1.49	0.00	4.51	1.05	0.24	0 12:17	0 00:00	0.00	0.00
14	V1	8.27	0.00	2.06	1.36	0.00	2.18	0.94	0.24	0 12:18	0 00:00	0.00	0.00

Mean Low Water Tide Scenario

Prepared by JL
Checked by GTT

Channel Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	E5Overland	0.40	0 12:19	7811.17	0.00	13.45	0.71	0.12	0.02	0.00		
2	NavyOverland	0.00	0 00:00	17830.75	0.00	0.00		0.00	0.00	0.00		
3	VerneerOverland	19.40	0 12:18	1807.37	0.01	1.88	3.01	0.24	0.19	0.00		

Mean Low Water Tide Scenario

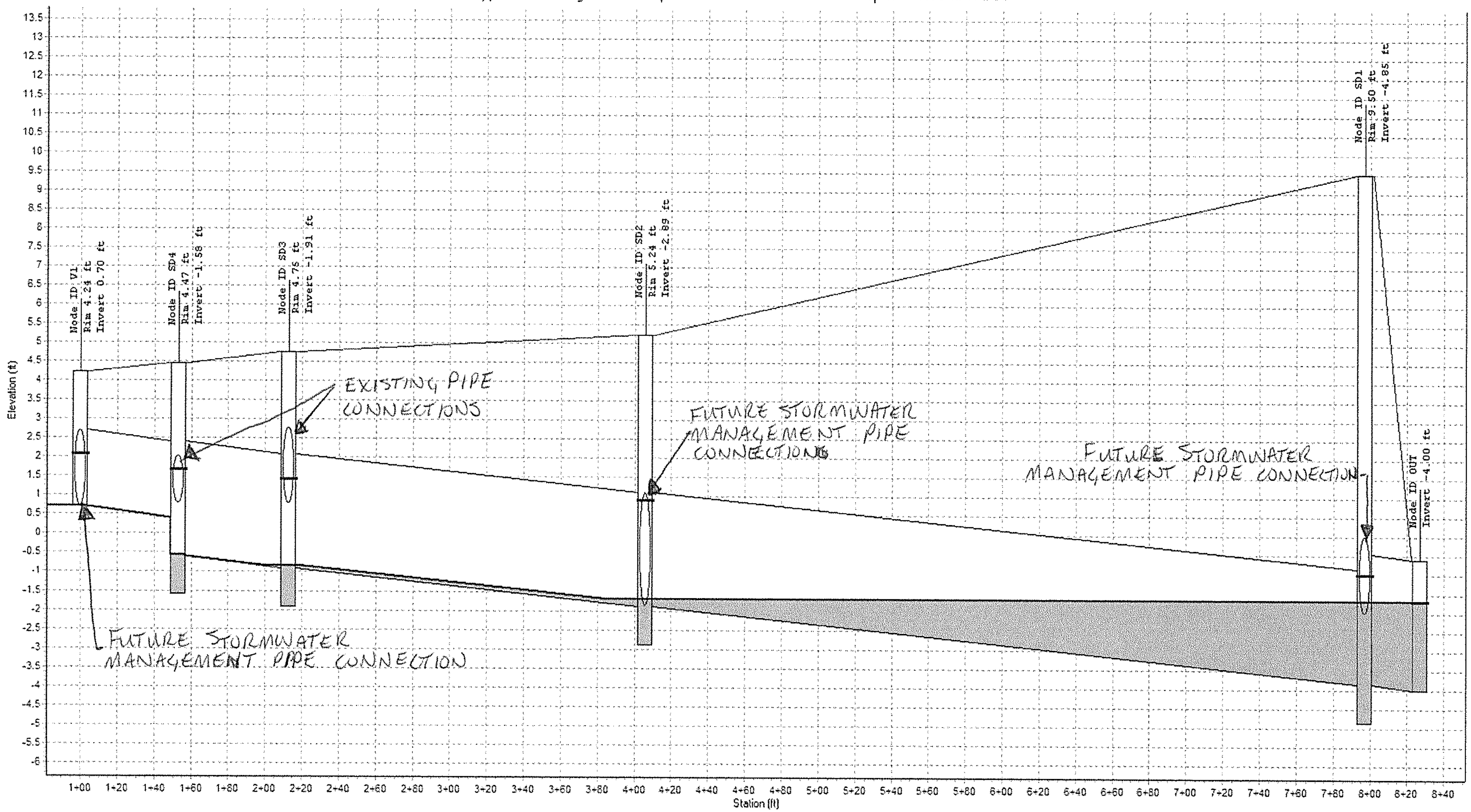
Prepared by JL
Checked by GAT

Pipe Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	E2-E1	24.44	0 12:17	8.28	2.95	7.99	0.62	1.87	0.94	0.00		> CAPACITY
2	E3-E2	24.44	0 12:17	2.85	8.58	7.78	0.16	2.00	1.00	30.00		SURCHARGED
3	E4-E3	6.86	0 12:26	4.57	1.50	2.18	1.10	2.00	1.00	31.00		SURCHARGED
4	E5-E4	6.86	0 12:26	6.88	1.00	3.88	2.96	1.50	1.00	23.00		SURCHARGED
5	E6-E5	1.61	0 12:09	5.39	0.30	1.57	1.02	1.25	1.00	22.00		SURCHARGED
6	E7-E6	1.50	0 12:12	0.83	1.80	2.95	0.24	1.00	1.00	17.00		SURCHARGED
7	J2-SD2	29.38	0 12:12	63.32	0.46	4.28	0.04	2.87	0.96	0.00		Calculated
8	SD1-OUT	124.37	0 12:12	216.82	0.57	5.58	0.09	2.58	0.76	0.00		Calculated
9	SD2-SD1	113.45	0 12:15	122.78	0.92	5.50	1.18	2.81	0.94	0.00		Calculated
10	SD3-SD2	86.69	0 12:19	102.98	0.84	7.34	0.44	2.55	0.85	0.00		Calculated
11	SD4-SD3	43.79	0 12:26	80.38	0.54	4.29	0.23	2.27	0.76	0.00		Calculated
12	SWM1-SD1	15.97	0 12:09	46.21	0.35	10.59	0.06	1.01	0.51	0.00		Calculated
13	SWM2-J2	14.59	0 12:12	35.55	0.41	3.23	0.57	2.31	0.93	0.00		Calculated
14	SWM3-J2	14.89	0 12:12	35.55	0.42	3.03	0.14	2.50	1.00	13.00		SURCHARGED
15	SWM4-V1	8.27	0 12:15	20.56	0.40	3.47	0.05	1.43	0.71	0.00		Calculated
16	V1-SD4	8.26	0 12:15	14.25	0.58	4.06	0.22	1.30	0.65	0.00		Calculated
17	V2-V1	8.19	0 12:18	2.25	3.64	6.81	0.83	1.19	0.95	0.00		> CAPACITY

Elm Avenue Storm Drain Relocation

Typical Standing Water in Pipe at Mean Low Water Initial Depth = -1.69 NAVD88



Mean Sea Level Tide Scenario

Prepared by JL
Checked by [illegible]

Junction Results

SN	Element ID	Peak Inflow (cfs)	Peak Lateral Inflow (cfs)	Max HGL Elevation Attained (ft)	Max HGL Depth Attained (ft)	Max Surcharge Depth Attained (ft)	Min Freeboard Attained (ft)	Average HGL Elevation Attained (ft)	Average HGL Depth Attained (ft)	Time of Max HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-in)	Total Time Flooded (min)
1	E2	24.44	0.00	5.42	4.29	0.00	0.81	1.63	0.50	0 12:16	0 00:00	0.00	0.00
2	E4	6.86	0.00	6.22	4.79	0.00	0.37	1.86	0.43	0 12:16	0 00:00	0.00	0.00
3	E6	1.50	0.00	7.58	3.09	0.00	0.36	4.62	0.13	0 12:18	0 00:00	0.00	0.00
4	E7	1.50	1.50	7.63	1.64	0.00	0.07	6.14	0.15	0 12:17	0 00:00	0.00	0.00
5	J2	29.58	0.00	1.85	3.62	0.00	2.38	-0.18	1.59	0 12:15	0 00:00	0.00	0.00
6	SD1	106.67	0.00	0.02	4.87	0.00	9.48	-0.24	4.61	0 12:13	0 00:00	0.00	0.00
7	SD2	98.16	0.00	1.60	4.49	0.00	3.64	-0.18	2.71	0 12:20	0 00:00	0.00	0.00
8	SD3	78.73	28.07	2.14	4.05	0.00	3.30	-0.17	1.74	0 12:21	0 00:00	0.00	0.00
9	SD4	42.81	12.88	2.30	3.88	0.00	2.17	-0.12	1.46	0 12:23	0 00:00	0.00	0.00
10	SWM1	15.97	15.97	1.09	1.09	0.00	3.71	0.15	0.15	0 12:09	0 00:00	0.00	0.00
11	SWM2	14.70	14.70	2.14	2.81	0.00	3.19	-0.17	0.50	0 12:15	0 00:00	0.00	0.00
12	SWM3	14.89	14.89	2.03	3.55	0.00	2.45	-0.17	1.35	0 12:15	0 00:00	0.00	0.00
13	SWM4	8.28	8.28	2.54	1.73	0.00	4.27	1.05	0.24	0 12:21	0 00:00	0.00	0.00
14	V1	8.26	0.00	2.43	1.73	0.00	1.81	0.95	0.25	0 12:22	0 00:00	0.00	0.00

Prepared by JL
Checked by GAT

Channel Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	E5Overland	0.40	0 12:19	7811.17	0.00	36.30	0.26	0.12	0.02	0.00		
2	NavyOverland	0.00	0 00:00	17830.75	0.00	0.00		0.00	0.00	0.00		
3	VerneerOverland	19.71	0 12:18	1807.37	0.01	1.39	4.08	0.56	0.43	0.00		

Mean Sea Level Tide Scenario

Prepared by JL
Checked by GAT

Pipe Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	E2-E1	24.44	0 12:17	8.28	2.95	7.99	0.62	1.87	0.94	0.00		> CAPACITY
2	E3-E2	24.44	0 12:17	2.85	8.58	7.78	0.16	2.00	1.00	30.00		SURCHARGED
3	E4-E3	6.86	0 12:26	4.57	1.50	2.18	1.10	2.00	1.00	31.00		SURCHARGED
4	E5-E4	6.86	0 12:26	6.88	1.00	3.88	2.96	1.50	1.00	23.00		SURCHARGED
5	E6-E5	1.61	0 12:27	5.39	0.30	1.57	1.02	1.25	1.00	22.00		SURCHARGED
6	E7-E6	1.50	0 12:12	0.83	1.80	2.95	0.24	1.00	1.00	17.00		SURCHARGED
7	J2-SD2	29.57	0 12:12	63.32	0.47	5.53	0.03	3.00	1.00	28.00		SURCHARGED
8	SD1-OUT	106.67	0 12:14	216.82	0.49	3.88	0.13	3.42	1.00	1440.00		SURCHARGED
9	SD2-SD1	98.17	0 12:20	122.78	0.80	4.63	1.41	3.00	1.00	29.00		SURCHARGED
10	SD3-SD2	79.05	0 12:27	102.98	0.77	5.72	0.56	3.00	1.00	7.00		SURCHARGED
11	SD4-SD3	50.60	0 12:33	80.38	0.63	4.60	0.22	2.94	0.98	0.00		Calculated
12	SWM1-SD1	15.96	0 12:09	46.21	0.35	6.13	0.10	1.54	0.77	0.00		Calculated
13	SWM2-J2	14.69	0 12:12	35.55	0.41	2.99	0.61	2.50	1.00	14.00		SURCHARGED
14	SWM3-J2	14.89	0 12:12	35.55	0.42	3.03	0.14	2.50	1.00	33.00		SURCHARGED
15	SWM4-V1	8.26	0 12:15	20.56	0.40	3.27	0.05	1.72	0.86	0.00		Calculated
16	V1-SD4	8.22	0 12:15	14.25	0.58	3.83	0.23	1.80	0.90	0.00		Calculated
17	V2-V1	8.28	0 12:15	2.25	3.68	6.80	0.83	1.25	1.00	16.00		SURCHARGED

Elm Avenue Storm Drain Relocation

Typical Standing Water in Pipe at Mean Sea Level Initial Depth = -0.25 NAVD88

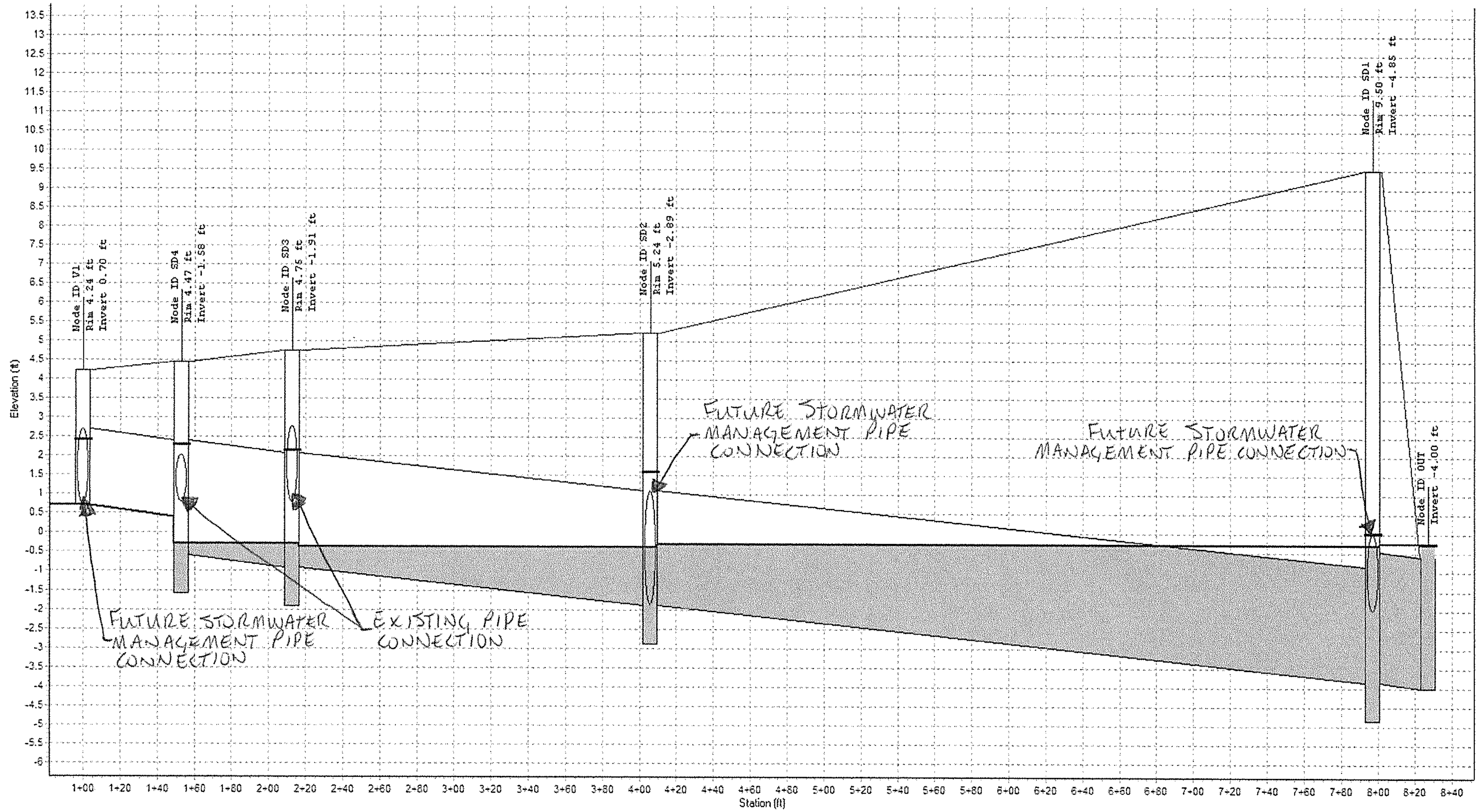


Table 1 - Hydraulic Modeling Summary

		Maximum Hydraulic Grade Elevation					Maximum Discharge at Outlet (cfs)
		SD Junction Box 1	SD Junction Box 2	SD Junction Box 3	SD Junction Box 4	Veneer Road Curb Inlet	
	Rim Elevation	9.50	5.42	4.75	4.47	4.24	
Mean High Water (MHW) 1.17 NAVD88	100year-24hour	1.57	3.69	4.32	4.47	4.24	130.0
	50year-24hour	1.53	3.52	4.13	4.47	4.24	124.0
	25year-24hour	1.49	3.32	3.94	4.47	4.24	115.6
	10year-24hour	1.44	3.05	3.61	3.81	3.98	106.5
	5year-24hour	1.40	2.79	3.28	3.44	3.57	98.2
	2year-24hour	1.27	1.85	2.03	2.09	2.16	64.7
Mean Sea Level (MSL) -0.25 NAVD88	100year-24hour	0.25	3.07	3.96	4.47	4.24	145.5
	50year-24hour	0.20	2.82	3.71	4.47	4.24	137.0
	25year-24hour	0.11	2.30	3.08	3.35	3.56	123.0
	10year-24hour	0.02	1.60	2.14	2.30	2.43	106.7
	5year-24hour	-0.01	1.41	1.77	1.91	2.09	101.1
	2year-24hour	-0.10	0.62	0.76	0.89	1.69	79.1
Mean Low Water (MLW) -1.69 NAVD88	100year-24hour	-0.72	2.59	3.71	4.47	4.24	154.1
	50year-24hour	-0.86	1.91	2.92	3.28	3.54	139.2
	25year-24hour	-0.92	1.28	1.92	2.16	2.39	132.5
	10year-24hour	-1.00	0.90	1.41	1.67	2.06	124.4
	5year-24hour	-1.08	0.64	1.24	1.49	1.94	116.0
	2year-24hour	-1.40	-0.23	0.63	0.82	1.69	78.9

* - Shaded values represent storm drain junctions functioning under flooded conditions.

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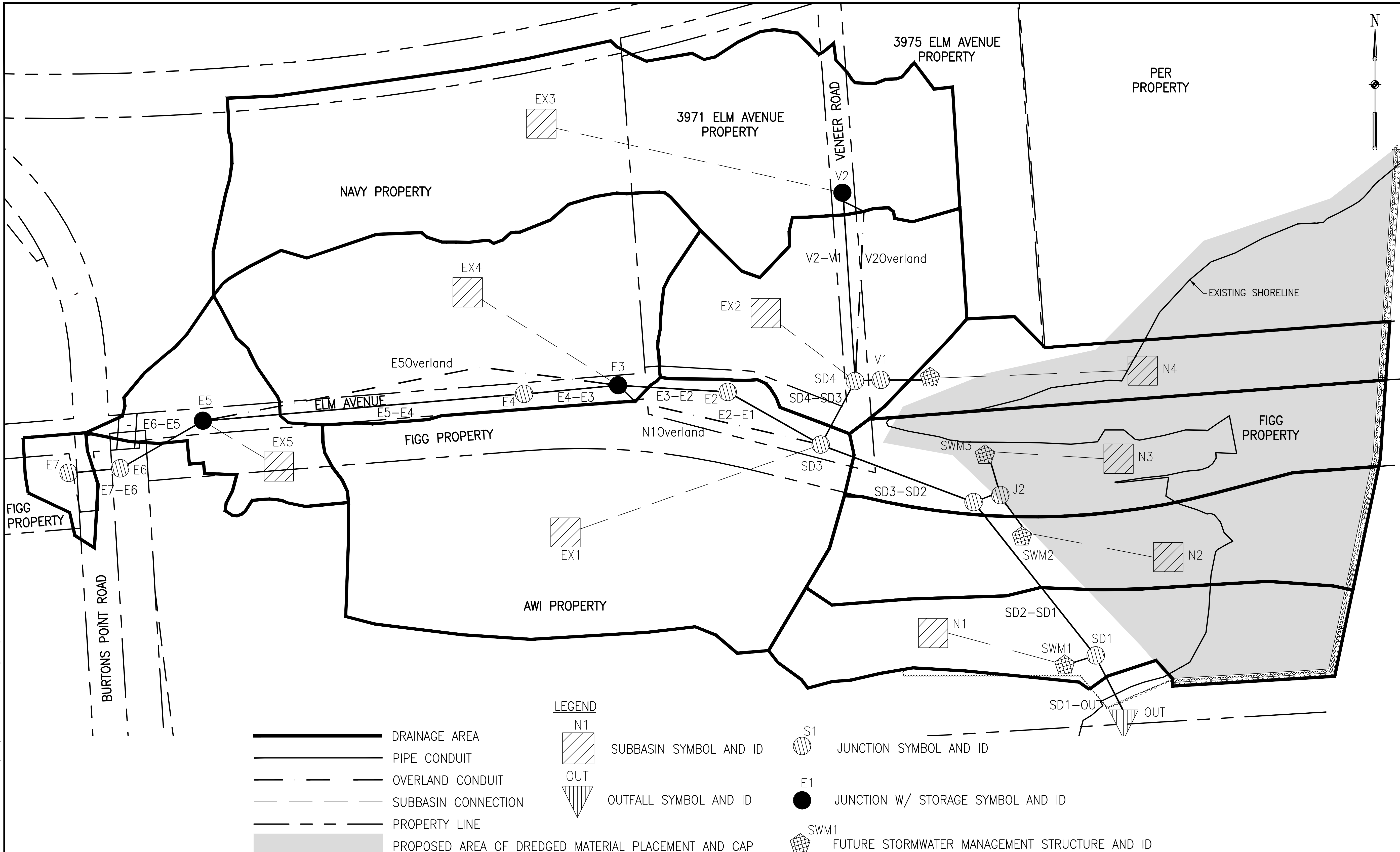

EA ENGINEERING, SCIENCE, AND TECHNOLOGY

DESIGNED BY	DRAWN BY	DATE	PROJECT NO.
JLL	JLL	MARCH 2012	1453011
CHECKED BY	PROJECT MGR.	DRAWING NO.	FIGURE
GAT	PAP	SCH1	4

REMEDIAL DESIGN
ELM AVEMUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT
PORTSMOUTH, VIRGINIA

POST DEVELOPMENT CONDITIONS MODEL SCHEMATIC

FILE PATH: L:\NON DOD\AWI CAD\PHASE 2\HYD\AWI SCHEMATIC.DWG [POST] 6/21/12

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Appendix F

Groundwater Analysis

- **Groundwater Model**
- **Groundwater Treatment Structures Technical Memo**
- **Mixing Zone**

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Groundwater Model

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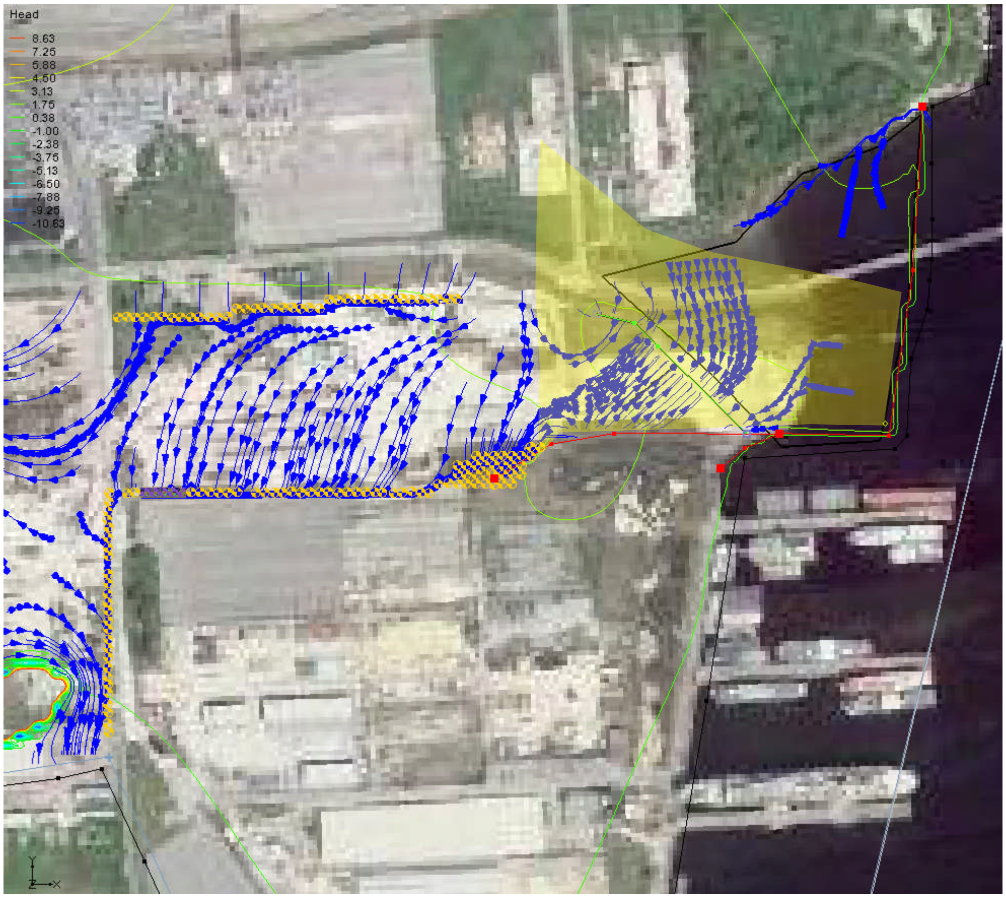
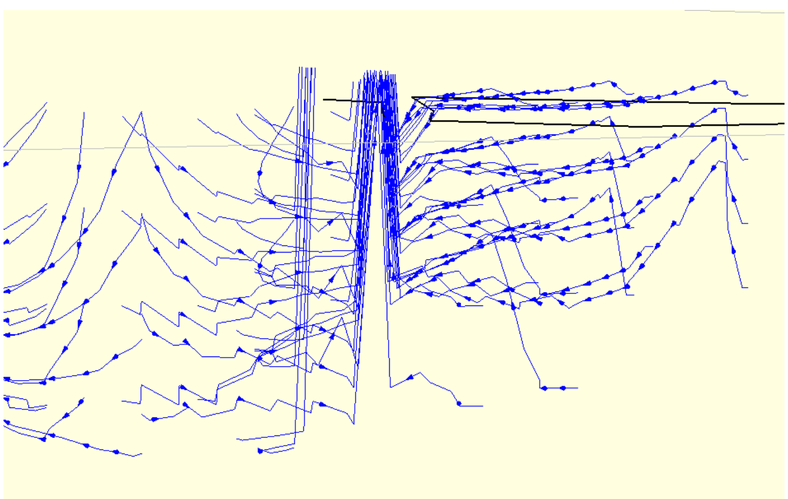
Figure 3 – Groundwater Flow Directions for Groundwater Trench at Elev. +1.5



Groundwater flow directions (blue arrows) in a modeled scenario with the OSPW (red line), dredged material cap (thick black line), trees for hydraulic control (yellow dots), and a groundwater trench at elevation +1.5 ft NAVD88 (green line just west of the dredged material cap). The yellow-shaded area shows the approximate area of groundwater that flows into the trench.

Figure 4 – Groundwater Flow Cross-Section for Groundwater Trench at Elev. +1.5



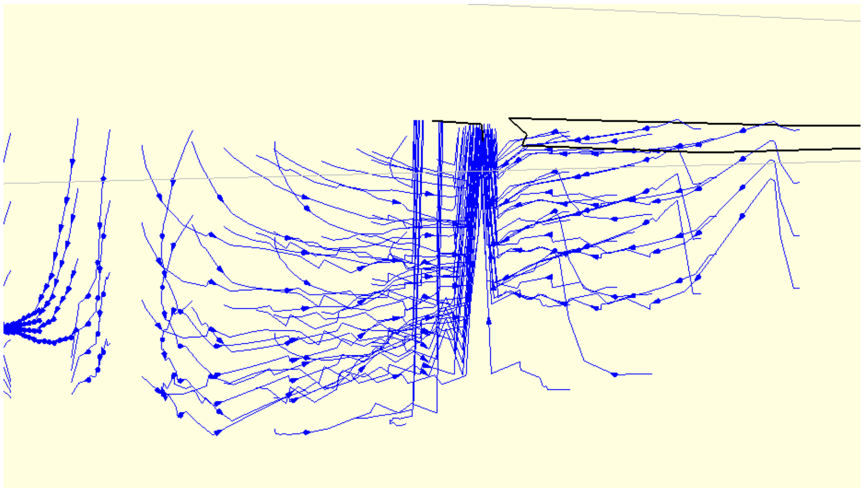
(NOTE: Vertical scale is exaggerated for display purposes.)

Model cross-section showing flow into the groundwater collection trench (shown as upward arrows along the trench) at elevation +1.5 ft, indicating that no water passes under the trench. Black polygon to the right (east) of the trench is the dredged material cap.

Figure 5 – Groundwater Flow Directions for Groundwater Trench at Elev. 0

Groundwater flow directions (blue arrows) in a modeled scenario with the OSPW (red line), dredged material cap (thick black line), trees for hydraulic control (yellow dots), and a groundwater trench at elevation 0.0 ft (green line just west of the dredge cap). The yellow-shaded area shows the approximate area of groundwater that flows into the trench.

Figure 6 – Groundwater Flow Cross-Section for Groundwater Trench at Elev. 0



(NOTE: Vertical scale is exaggerated for display purposes.)

Model cross-section showing flow into the groundwater collection trench (shown as upward arrows along the trench) at elevation 0 ft, indicating that no water passes under the trench. Black polygon to the right (east) of the trench is the dredged material cap.

Groundwater Treatment Structures
Technical Memorandum

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Project: Atlantic Wood Industries (AWI) Superfund Site – Remedial Design
Topic: **Groundwater Treatment Structures**
Pre-Final Design Technical Memorandum
Date: 22 March 2012

EA Engineering, Science, and Technology, Inc. (EA) has prepared this Groundwater Treatment Structures Pre-Final Design Technical Memorandum (TM) for the design of the future groundwater treatment vaults which may be required to treat groundwater from the groundwater collection trench prior to discharge to the Elizabeth River.

The Preliminary Design included a flat, stone trench to collect and convey groundwater. This design was based on a modeling effort performed as part of the preliminary submittal in December 2011. This modeling effort, described in the Preliminary BOD, resulted in an estimated maximum groundwater flow of 5 gallons per minute (gpm) through the trench. This water will be collected in a 6-inch HDPE pipe and conveyed to the treatment portion of the design.

The Preliminary Basis of Design included two vaults for possible groundwater treatment and monitoring, placed in series. The intent was that these vaults would contain passive treatment media in order to minimize operations and maintenance efforts and costs.

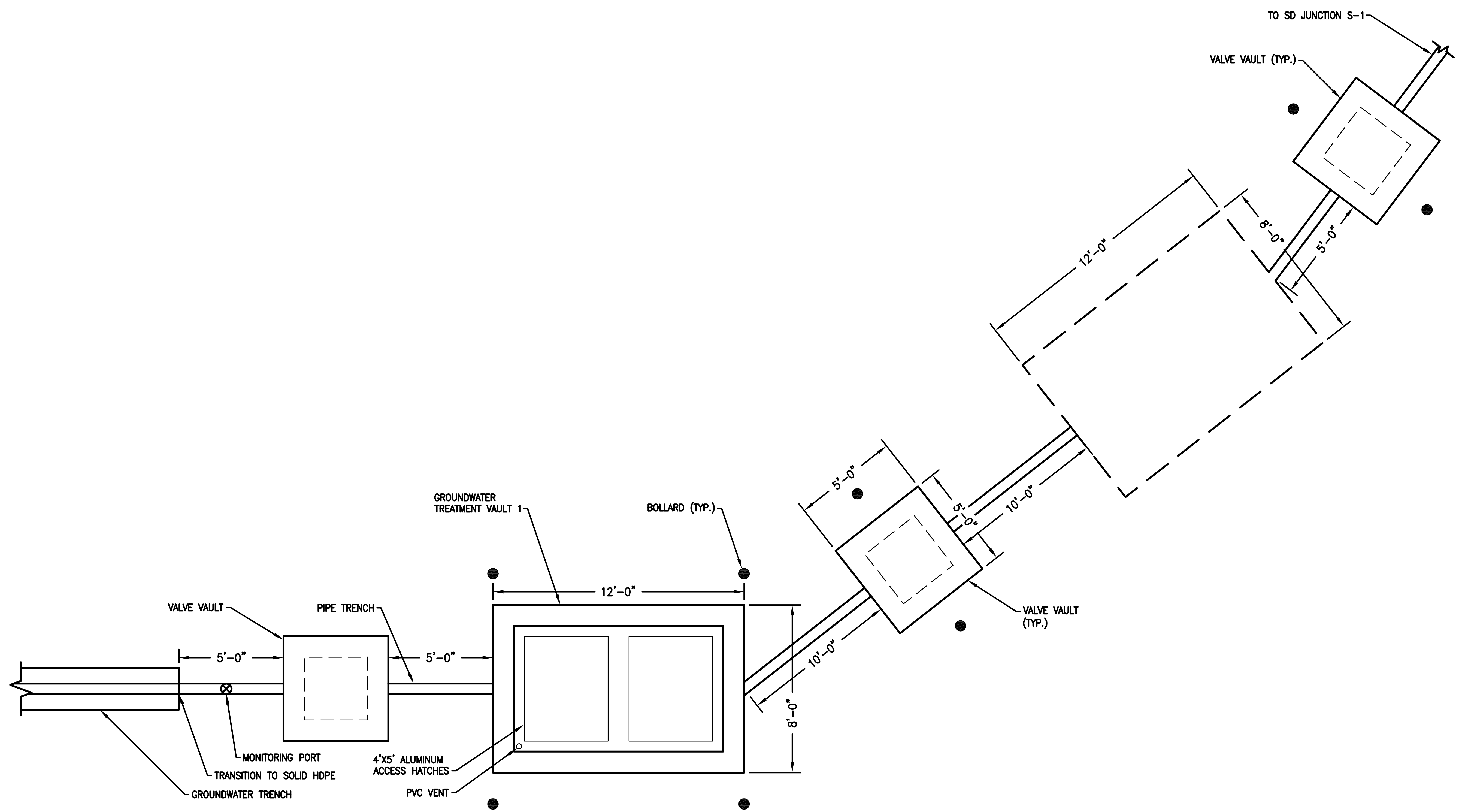
For the Pre-Final Design it was further determined that one treatment media would likely be required for organics removal and one treatment media would be required for metals removal. Effort for this design submittal included attempting to determine which of the treatment methods outlined in the Groundwater Alternatives Analysis would be most feasible, and if the vaults, which were not sized in the Preliminary design, could be sized for organics and metals treatment based on such a media selection. The design was modified to include valve vaults to control flow into each of the treatment vaults. Influent and effluent elevations for each treatment vault were determined based on the placement of the trench (+1 ft NAVD 88) and the elevation of the discharge into Stormwater Junction Box 1 (-1.77 ft NAVD 88). Figure 1 provides a plan view of the proposed groundwater treatment vaults with discharge to Stormwater Junction Box 1. The discharge to Stormwater Junction Box 1 is to have a 6-inch pinch/tidal valve which is to prevent back flow into the groundwater management system when elevations within the Stormwater Junction Box 1 are greater than -1.77 ft NAVD88.

In order to accurately size the groundwater treatment vaults, the existing groundwater data were evaluated. During the evaluation, only the data from the wells in the trench watershed were considered, since only the water in that area would be flowing through the groundwater collection trench and directed to the treatment vaults. Effluent criteria were assumed to be equal to Virginia Water Quality Standards Surface Water Discharge Limits and/or Hampton Roads Sanitation District, whichever is more stringent.

During the evaluation of groundwater chemistry data, the groundwater treatment vault for the organics was conservatively sized based on preliminary information using granular activated carbon. The groundwater treatment vault was sized to hold what was estimated to be sufficient media to reduce media maintenance and change-out (Figure 2). Valve vaults were placed before and after the groundwater treatment vault with gate valves to discontinue flow during periods of maintenance (Figure 3).

The groundwater treatment vault for the metals was not sized. Virginia's mixing zone requirements (9VAC25-260-20) for a discharge to estuarine waters were first reviewed and discharge was modeled to determine if the result of the mixing zone would allow for the metals to not undergo separate treatment. Dilution factors were determined for several scenarios, including only collected groundwater discharge and combined stormwater/groundwater discharge and are included as Attachment 1. Mixing zones may be used in the NPDES process to calculate end-of-pipe permit limits to ensure protection of resident aquatic life. More specifically, the dilution factors determined using state mixing zone guidance are used in combination with numeric ambient water quality standards to calculate end-of-pipe permit limits (9VAC25-260-20). As an example, Virginia DEQ's acute and chronic ambient water quality criteria (WQC) for copper in estuarine waters are 9.3 µg/L and 6.0 µg/L, respectively. If, for example, a mixing zone dilution factor of 5.4 can be supported, then end-of-pipe permit limits for copper could be 50.2 µg/L as a daily maximum and 32.4 µg/L as a monthly average. Similar benefits could be obtained for other regulated chemical constituents in the discharge (Permit Limit = WQC x DF). It will be important to discuss the use of mixing zones with VDEQ permitting staff to determine the Department's acceptance given the variable and intermittent discharge conditions at the AWI site. The outcome of that discussion could substantively affect facility-specific wastewater treatment requirements.

Due to the number of constituents and the range of concentrations associated with historical monitoring data, it was determined that the volume and frequency of media replacement to treat groundwater at the site effectively could not be determined. While the treatment methods identified in the Alternatives Analysis are all viable methods for treating contaminants such as are present at AWI, further analysis is required to determine the most feasible method, and to estimate treatment media volume needed. **Therefore, the future valve vaults are not included in the design documents for the Elm Avenue Storm Drain Relocation.**



PRE-FINAL DESIGN

EA ENGINEERING, SCIENCE, AND TECHNOLOGY	DESIGNED BY JLL	DRAWN BY DPA	DATE 3-15-2012	PROJECT NO. 14530.11	REMEDIAL DESIGN ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT ATLANTIC WOOD INDUSTRIES SUPERFUND SITE PORTSMOUTH, VIRGINIA	FIGURE 1 PROPOSED GROUNDWATER TREATMENT VAULTS
	CHECKED BY GAT	PROJECT MGR. PAP	SCALE 1" = 5'	FIGURE 1		

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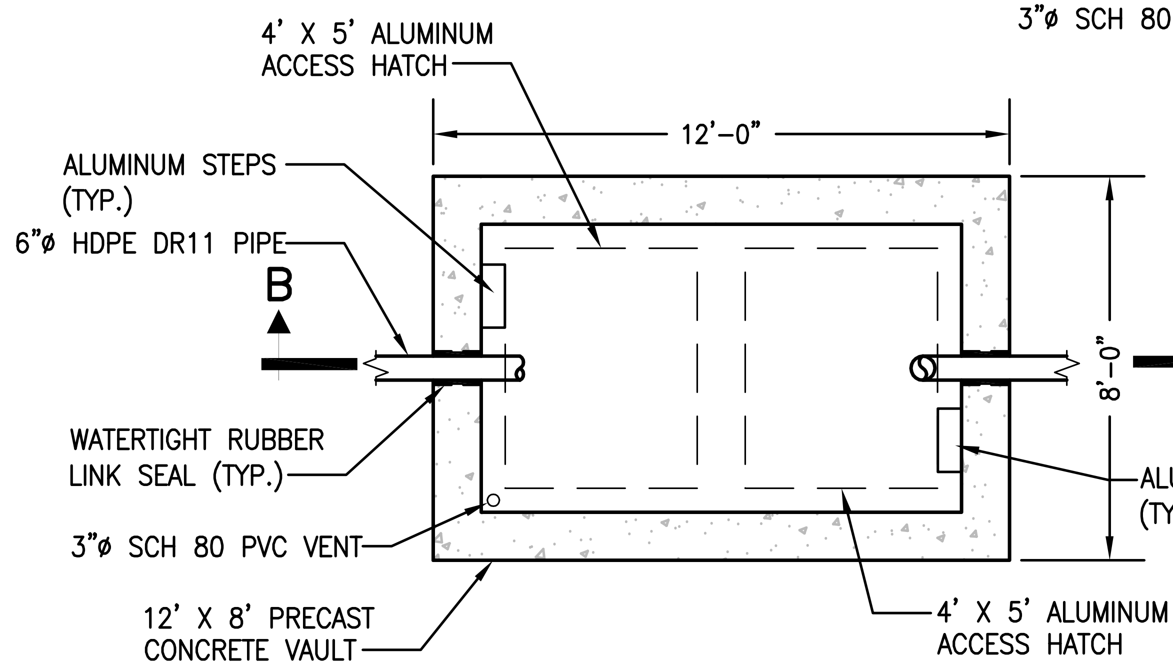
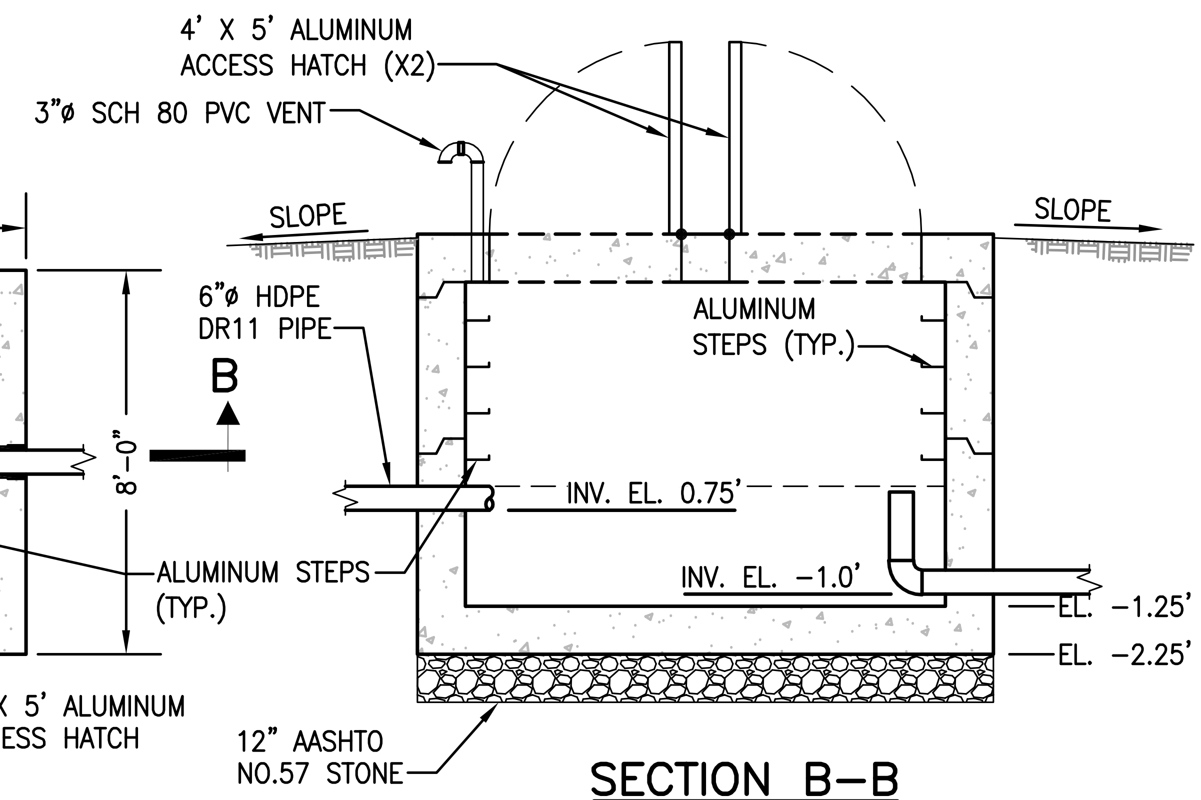
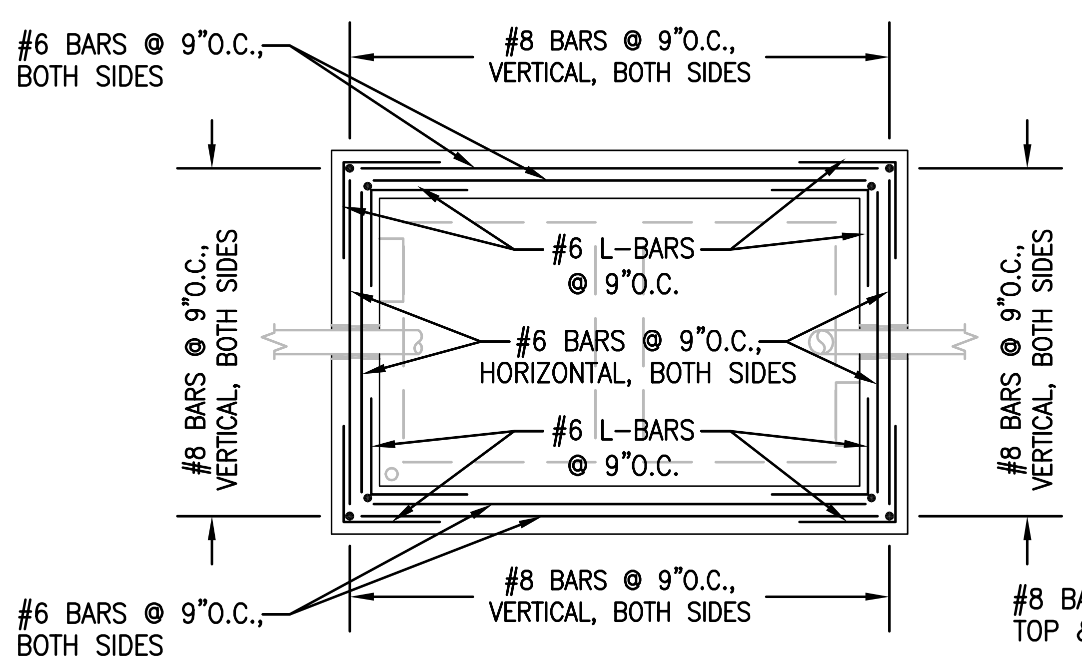


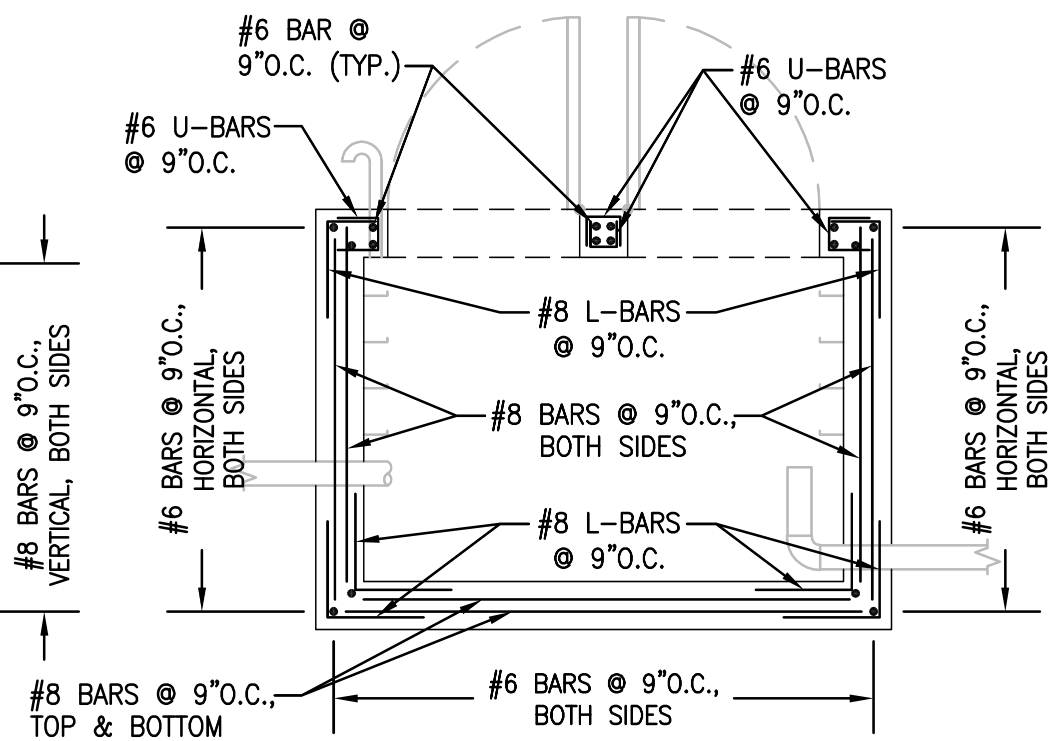
PLAN VIEW



SECTION B-B



PLAN VIEW REBAR



SECTION B-B REBAR

PRE-FINAL DESIGN

EA® EA ENGINEERING, SCIENCE, AND TECHNOLOGY	DESIGNED BY JLL	DRAWN BY DPA	DATE 3-15-2012	PROJECT NO. 14530.11	REMEDIAL DESIGN ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT ATLANTIC WOOD INDUSTRIES SUPERFUND SITE PORTSMOUTH, VIRGINIA	FIGURE 2 GROUNDWATER TREATMENT VAULT 1
	CHECKED BY GAT	PROJECT MGR. PAP	SCALE 1/4" = 1'	FIGURE 2		

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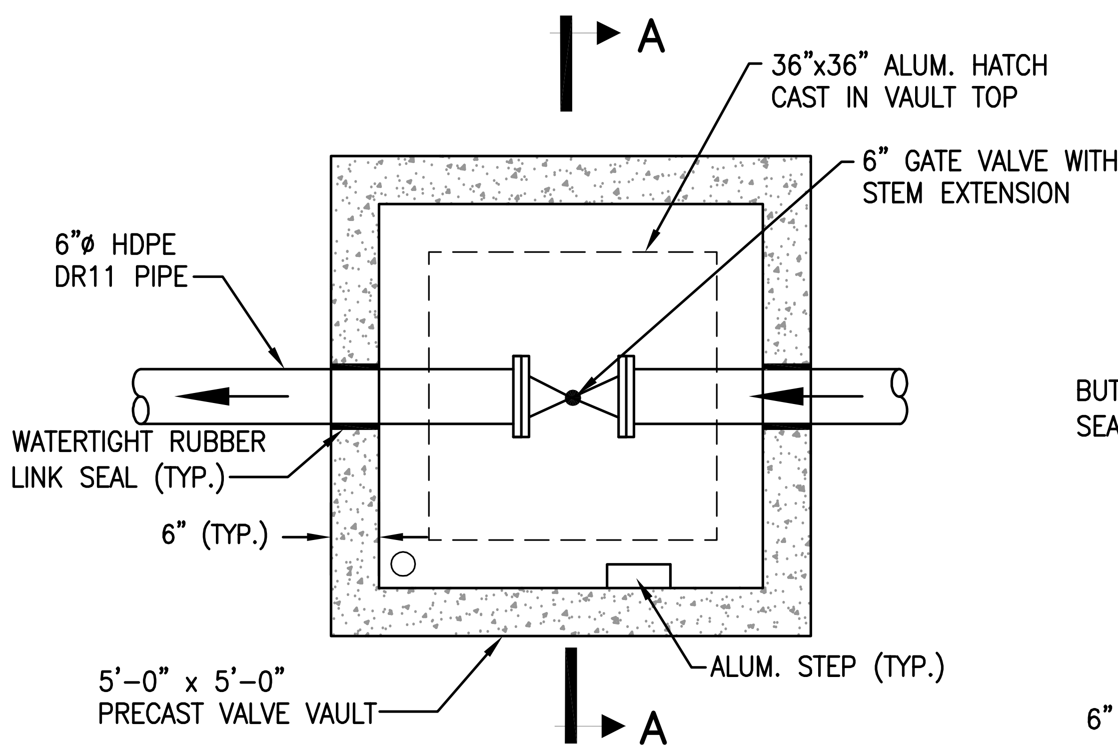
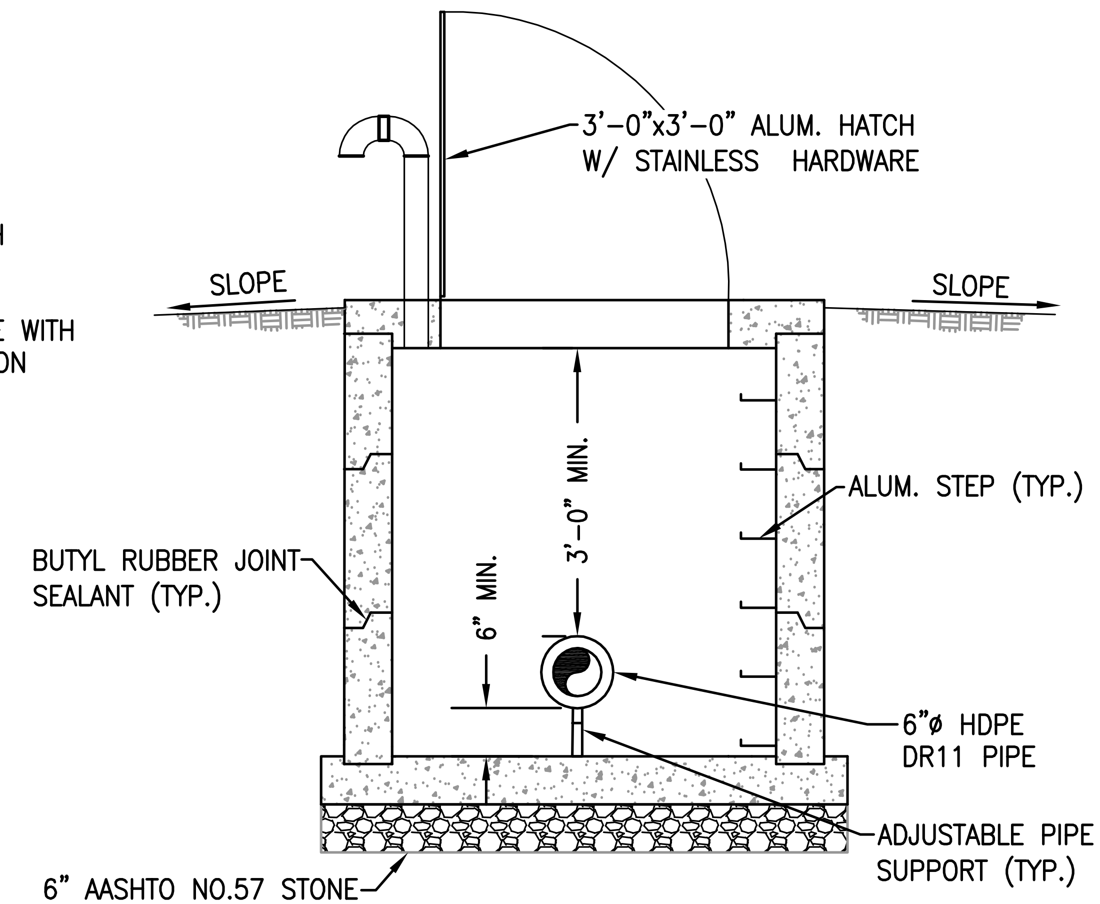


PLAN VIEW



SECTION A–A

PRE-FINAL DESIGN

EA ENGINEERING, SCIENCE, AND TECHNOLOGY	DESIGNED BY: JLL	DRAWN BY: DPA	DATE: 3–15–2012	PROJECT NO.: 14530.11	REMEDIAL DESIGN ELM AVENUE STORM DRAIN RELOCATION AND GROUNDWATER MANAGEMENT ATLANTIC WOOD INDUSTRIES SUPERFUND SITE PORTSMOUTH, VIRGINIA	FIGURE 3 VALVE VAULT DETAIL
	CHECKED BY: GAT	PROJECT MGR.: PAP	SCALE: 1/2" = 1'	FIGURE: 3		

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Attachment 1

Mixing Zone Calculations

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Mixing Zone Calculations for Atlantic Wood Industries (AWI) Superfund Site

The proposed discharge for the site consists of three 42 inch (3.5 ft) pipes. The top of the pipes are 0.25 ft below mean sea level (MSL). At mean low water (MLW), 1.19 ft of the pipes would be exposed. The discharge will be placed at a shoreline bulkhead. Flow scenarios include a 148 cfs peak stormwater flow and a 5 gpm (0.011 cfs) groundwater flow.

Principle components of Virginia's mixing zone requirements (9VAC25-260-20) for a discharge to estuarine waters are summarized as follows:

- Shall not extend more than five times in any direction the average depth of the receiving water.
- A subsurface diffuser shall be required for any new or expanded freshwater discharge greater than 0.5 mgd (0.77 cfs) to estuarine waters.
- The acute and chronic criteria shall be met at the edge of the zone of initial mixing. The zone of initial mixing is the area where mixing of ambient water and effluent is driven by the jet effect and/or momentum of the effluent.

The local post-dredging depth at the proposed discharge location is approximately 28 ft. Any proposed mixing zone would be will within five times this distance. The proposed discharge is fully submerged over slightly more than one-half of the tidal cycle. This might potentially meet the subsurface requirement. The total discharge port area of three 42 inch pipes is 28.8 ft^2. At the peak 148 cfs stormwater flow, this total port area results in a 5.1 ft/sec exit velocity. At a 50 cfs stormwater flow, the exit velocity would decrease to 1.7 ft/sec. At the higher stormwater flows, the associated exit velocities would provide an initial momentum based mixing region. The extent of this region decreases with decreasing flow. At the 0.011 cfs groundwater flow, the exit velocity for the current discharge configuration would be nil (0.00038 ft/sec), and a zone of initial mixing may not exist per VDEQ guidance.

The proposed and an alternative discharge configuration were modeled with CORMIX ver7.0. The CORMIX model consists of a series of modules for various stages in the mixing process and model output indicates the end of the initial mixing region location. State regulatory agencies generally accept the location indicated by CORMIX as meeting their momentum based mixing zone criteria.

CORMIX requires several site characteristics. The depth of the receiving water was set as 28 ft, the proposed post-dredging depth at the shoreline bulkhead where the outfall pipes will be located. The receiving water is brackish and a 10 ppt salinity was assumed. The freshwater discharge will result in a buoyant plume. Site specific receiving water velocities are not readily available. The discharge configuration may be in the lee during an ebbing tide. Scenarios for permitting generally include a near slack water scenario for worst case. In the absence of site specific receiving stream velocity data, the model was executed for a range of relatively low velocities of 0.065 ft/sec, 0.16 ft/sec and 0.33 ft/sec (2 cm/sec, 5 cm/sec, 10 cm/sec).

CORMIX model results at the end of the initial mixing region for a range of stormwater flows and receiving stream velocities are provided in the following table.

Predicted Stormwater Dilution Factors for Proposed Discharge
at Three Different Receiving Stream Velocities

Flow (cfs)	0.065 ft/sec	0.16 ft/sec	0.33 ft/sec
148	9.1	11.3	17.2
100	6.2	7.5	12.3
70	4.5	5.2	8.6
40	nv	4.0	7.4

The CORMIX model would not execute for the 40 cfs/0.065 ft/sec scenario. The above table indicates that the available dilution factor at the end of the initial mixing region decreases with decreasing effluent flow and decreasing receiving water velocity.

The 5 gpm (0.011 cfs) groundwater flow scenario was executed in CORMIX for the proposed three 42 inch pipes. For this scenario, there was no initial mixing region present in the model since the extremely low exit velocity provided no initial momentum jet. However, the model does predict dilution that occurs as the effluent mixes into the receiving water. The resulting dilution factors as a function of the radial distance from the discharge are provided in the following table for a range of receiving stream velocities.

Predicted Groundwater Dilution Factors for Proposed Discharge
at Three Different Receiving Stream Velocities

Distance (ft)	0.065 ft/sec	0.16 ft/sec	0.33 ft/sec
2	1.0	1.0	1.0
5	1.0	1.0	1.0
10	1.4	1.7	1.7
15	2.2	2.9	3.1
20	3.0	4.2	5.1
30	4.7	6.7	10.5
40	6.3	9.2	18.3

A series of smaller pipe sizes were examined with CORMIX to determine a discharge configuration for the 5 gpm groundwater flow that would provide an initial mixing region. Initial mixing regions were found to exist for pipe diameters less than 2.5 inches. For a 2.0 in. diameter pipe, the exit velocity is 0.50 ft/sec and for a 1.5 in. pipe, the exit velocity is 0.90 ft/sec. The following tables provide predicted dilution factors for two small pipes and the distance CORMIX indicates as the end of the initial mixing region.

Predicted Groundwater Dilution Factors for Small Pipe

Diameter (in.)	0.065 ft/sec	0.16 ft/sec	0.33 ft/sec
2.0	16.6	3.6	5.8
1.5	28.9	7.4	6.5

Predicted Distance to End of Initial Mixing Region

Diameter (in.)	0.065 ft/sec	0.16 ft/sec	0.33 ft/sec
2.0	64	24.5	11.6
1.5	84	32.1	15.4

The above tables indicate that CORMIX predicted dilution factors at the end of an initial mixing region for a small pipe ranged from 3.6 to 28.9. At a 0.16 ft/sec receiving water velocity, the 3.6-7.4 dilution factors occurred at 24-32 ft distances. CORMIX modeling of the 5 gpm groundwater discharge from the proposed three 42 inch pipes predicted similar dilution factors at similar distances, even though an initial mixing region may not be present.

Mixing zones may be used in the NPDES process to calculate end-of-pipe permit limits to ensure protection of resident aquatic life. More specifically, the dilution factors determined using state mixing zone guidance are used in combination with numeric ambient water quality standards to calculate end-of-pipe permit limits (9VAC25-260-20). As an example, Virginia DEQ's acute and chronic ambient water quality criteria (WQC) for copper in estuarine waters are 9.3 µg/L and 6.0 µg/L, respectively. If, *for example*, a mixing zone dilution factor of 5.4 can be supported, then end-of-pipe permit limits for copper could be 50.2 µg/L as a daily maximum and 32.4 µg/L as a monthly average. Similar benefits could be obtained for other regulated chemical constituents in the discharge (Permit Limit = WQC x DF). It will be important to discuss the use of mixing zones with VDEQ permitting staff to determine the Department's acceptance given the variable and intermittent discharge conditions at the AWI site. The outcome of that discussion could substantively affect facility-specific wastewater treatment requirements.

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Appendix G

Geotechnical Report

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GEOTECHNICAL ENGINEERING REPORT

Elm Avenue Storm Drain Relocation (EASDR)
Atlantic Wood Industries (AWI) Superfund Site
Elm Avenue and Veneer Road
Portsmouth, Virginia

Schnabel Reference # 10233031.05
June 21, 2012

Prepared For:

EA Engineering, Science and Technology, Inc.

Schnabel
ENGINEERING

Schnabel
ENGINEERING

June 21, 2012

(b) (4) [redacted]
EA Engineering, Science and Technology, Inc.
One Marketway West, Suite 4C
York, PA 17401

Subject: **Project 10233031.05, Geotechnical Engineering Report, Elm Avenue Storm Drain Relocation (EASDR), Atlantic Wood Industries (AWI) Superfund Site, Elm Avenue and Veneer Road, Portsmouth, Virginia**

Dear Mr. Pellissier:

SCHNABEL ENGINEERING, LLC (Schnabel) is pleased to submit our geotechnical engineering report for this project. This document includes attached figures, tables, and appendices with relevant data collected for this study. This study was performed in accordance with our proposal dated March 29, 2012, as authorized by Modification 4 to Subcontract No. 6866 with Schnabel Engineering, executed on April 26, 2012.

SCOPE

Our agreement dated March 29, 2012, defines the scope of this study. Our services include subsurface exploration, field engineering, soil laboratory testing, and development of geotechnical engineering recommendations. The objective of this study is to evaluate the subsurface conditions and provide recommendations regarding the design of foundations, earthwork, and construction considerations for this project.

Services not described in our agreement are not included in this study. We would be happy to provide additional support services to the design team as the project demands.

SITE DESCRIPTION

The AWI (now Atlantic Metrocast, Inc.) site is located on the western shoreline of the Southern Branch of the Elizabeth River, generally south of the PER property and north of the Norfolk Naval Base Southgate Annex. This contaminated site, including an area of the Elizabeth River adjacent to the site, contains wastes from wood treatment operations, abrasive blast media waste and acetylene sludge waste. Part of the remedial design for site reclamation includes an off shore sheet pile containment system planned for construction in the Elizabeth River to contain dredged contaminated river sediments. The site for this project includes the Atlantic Metrocast property generally south of Elm Avenue to the south property boundary and east of Veneer Road extended to the Elizabeth River. Grades in the area vary from about

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300 Ed Wright Lane, Suite I / Newport News, VA / 23606

El 1 near the southern perimeter of the site along the river, to about El 5 near Elm Avenue at Veneer Road.

The area occupied by AWI east of Burton's Point Road consists of about 20 acres of land used for precast concrete product fabrication. The ground surface in this area consists of crushed stone, concrete dust, concrete waste material, and other low-permeability material. This area has been built up over the years with placement of these materials, based on information provided by AWI personnel. The eastern area of the site has also been used as a staging area by a marine contractor.

We obtained the site information from the topographic design site plan dated March, 2012, prepared by EA Engineering, Science and Technology, and through our site visits. A vicinity map is included as Figure 1.

PROPOSED CONSTRUCTION

This project includes the relocation of an existing storm drain to a new river discharge point due to the construction of the offshore sheet pile containment system. The relocated storm drain is to begin at a new junction box near the intersection of Veneer Road and Elm Avenue and extend about 700 feet southeast to a new discharge structure. The proposed storm drain will consist of two and three-36 inch diameter reinforced concrete storm drain pipes installed in a lined trench constructed with new uncontaminated structural backfill after the excavation and removal of existing contaminated soils. The roughly 14 to 20 ft wide trench will be lined with a 40 mil liner of polyethylene geomembrane, and the pipes will rest on a 6 inch thick bed of graded aggregate. The drain pipes will be set at a 0.5% grade sloping towards the discharge point. The finish grade for the ground surface above the pipes is expected to be at El 7. Trench depths will vary up to about 10 ft. The top one foot of compacted trench backfill is to consist of a low permeability CR-6 (dense graded crusher run aggregate) material.

Junction boxes up to about 32 ft wide and 15 ft long are to be used at major pipe junctions or turns. The reinforced concrete boxes will have floor slabs and walls up to 3 ft thick and 1.5 ft thick roofs. The junction box subgrades may be up to about 12 ft below grade with roof slabs at grade.

Mobile gantry cranes are used by Atlantic Metro Cast, Inc. to transport fabricated precast concrete products at the site. We understand that bollards will be placed around the junction boxes to prevent these cranes from traversing over the structures. The cranes may traverse the pipe trenches. Ground contact pressures of between 78 psi and 120 psi are possible under the wheels.

The project details were obtained from The Pre-Final Design plan drawings by EA Engineering, Science and Technology, dated March of 2012.

SUBSURFACE CONDITIONS

Geology

We reviewed existing geologic data and information in our files. Based on this review, the geologic stratigraphy consists of recent alluvial soil deposits overlying the Pleistocene Age alluvial soils of the Norfolk Formation. These soils typically overlay alluvial soils of the Miocene Age Yorktown Formation, which was not encountered in the borings drilled for this project. The Norfolk Formation is part of the Columbia Group; previous investigations have referred to these sediments as undifferentiated deposits of the Columbia Group. The Norfolk Formation typically consists of cross-bedded fine and coarse-grained soils. The fine-grained soils generally consist of clays and silts containing varying amounts of sand. These soils are generally normally consolidated to slightly preconsolidated. The coarse-grained soils generally consist of poorly graded sands, silts, silty sands and clayey sands, and may contain gravel.

Data Collection Techniques

We performed test borings and soil laboratory testing on samples collected to develop our geotechnical recommendations. Appendix A includes our summary of soil laboratory test results and laboratory test curves. Appendix B includes the logs from our subsurface exploration.

Our geotechnical laboratory conducted tests on selected samples obtained in the borings. This testing aided in the classification of soils encountered in the subsurface exploration, and provided data for use in the development of our recommendations. The logs in Appendix B show the natural moisture content values of selected soil samples. Appendix A presents the results of the remaining laboratory tests.

Fishburne Drilling, Inc., of Chesapeake, Virginia, drilled four borings at this site under our observation on May 4 and 5, 2012. In addition, one additional test boring drilled (9/26/2008) for earlier studies on this site by Schnabel has also been included for informational purposes. Appendix B includes specific observations, remarks, and logs for the borings; classification criteria; and sampling protocols. Figure 2 shows the approximate boring locations. The locations and elevations from the field are tabulated below. We will retain soil samples up to 45 days beyond the issuance of this report, unless you request other disposition.

BORING LOCATIONS AND ELEVATIONS

BORING	NORTHING	EASTING	ELEV
B-101	3461122.90	12128302.07	4.8
B-102	3461070.58	12128496.99	3.2
B-103	3460986.09	12128566.16	7.5
B-104	3460861.34	12128669.37	6.6

Throughout field geotechnical operations, proper health and safety procedures were followed in accordance with EA's Health and Safety Plan (HASP), including the PPE and environmental monitoring. No health and safety incidents occurred during field activities. All test borings were backfilled with a cement/bentonite grout.

Generalized Subsurface Stratigraphy

We have characterized the following generalized subsurface soil stratigraphy based on the boring and laboratory data presented in Appendix B:

Ground Cover:

Borings B-101 and B-102 contained up to 0.3 ft of rootmat and topsoil. Boring B-103 was overlain with approximately 0.5 ft of crushed stone and concrete.

Stratum A: Existing Fill

Existing FILL soils, denoted as Stratum A, were encountered in all four of the borings from the ground surface to depths of 2.0 to 8.0 ft. The fill consisted of fine to coarse grained Silty Sand, containing varying amounts of roots, peat, shells, brick fragments, concrete, crushed stone and wood. Standard Penetration Test N-values ranged from 3 to 21, indicating very loose to very dense soils. Borings B-103 and B-104 encountered 2 to 4 ft of dense, creosote-treated wood.

Stratum B: Recent Alluvium

Below the fill soils of Stratum A, the borings encountered a deposit of recent alluvium consisting of gray to greenish gray, fine to coarse grained SILTY SAND, CLAYEY SAND, POORLY GRADED SAND with SILT, and LEAN CLAY (SC, SM, SP-SM, CL) with varying amounts of organics and shells to depths of 4 to 13 ft, El 0.8 to -5.5. Based on the Standard Penetration Tests performed, this stratum is generally loose to medium density: N = 4 to 13.

Stratum C: Norfolk Formation

Below the Alluvial soils of Stratum B, the borings encountered the cross-bedded fine and coarse grained soils of the Pleistocene Age Norfolk Formation to depths of 30 to 32 ft, El -22.5 to -26.8, the maximum depths of penetration. The coarse grained soils of the Norfolk Formation, identified as Stratum C1, consisted of SILTY SAND (SM), and POORLY GRADED SAND with SILT (SP-SM). The coarse grained soils were generally overlain atop the fine grained soils, with the exception of Boring B-101, where the fine and coarse grained soils were inter-layered. The SPT N-values of the coarse grained soils ranged from 2 to 8, indicating very loose to loose density soils. The average moisture content of the coarse grained soils ranged from 21.3 to 30.6 percent, with an average value of 24.9 percent. The coarse grained soils were non-plastic.

The fine grained Norfolk Formation soils, identified as Stratum C2, consisted of FAT CLAY (CH) with varying amounts of sand, shells, and organics. The Standard Penetration Test (SPT) N-values ranged from WOH/24” to 2, indicating very soft to soft consistency soils. The natural moisture content of the fine grained soils ranged from 59.3 to 77.1 percent, with an average moisture content of 68.7 percent. These soils were generally of very high plasticity having liquid limits between 69 and 76, and plasticity indices of 41 to 45.

Two consolidation tests were performed on the fine grained soils of Stratum C. The consolidation test indicated that the soils were normally consolidated to slightly pre-consolidated to about 0.17 to 0.2 tsf in excess of the present overburden pressure. The remaining test results are summarized in Appendix A and on the test boring logs in Appendix B.

Groundwater

The logs note groundwater level readings obtained in the borings during and after completion. We obtained groundwater level readings in open boreholes after completion at depths of 1.7 to 6.1 ft, El 3.1 to 0.2. These levels may or may not represent stabilized water level readings as the borings were backfilled upon completion for safety.

Our drilling subcontractor installed groundwater observation wells in Borings B-101, B-102, and B-104. We recorded groundwater levels in the wells at depths of 1.0 ft to 4.9 ft, El 3.7 to 2.0, 5 days after completion of the drilling. After final readings, the wells were pulled and the borings backfilled with grout.

The groundwater levels on the logs show our estimate of the hydrostatic water table at the time of drilling. The final design should anticipate fluctuations in the hydrostatic water table depending on variations in precipitation, surface runoff, pumping, river levels, evaporation, leaking utilities, and similar factors.

GEOTECHNICAL RECOMMENDATIONS

We based our geotechnical engineering analysis on the information developed from our subsurface exploration and soil laboratory testing, along with the project development plans, site plans, and structural loading furnished to our office. The following sections of the report provide our detailed recommendations.

The site is underlain with normally to slightly preconsolidated fine-grained alluvial soils of the Norfolk Formation. These soils were generally encountered at or just below the proposed construction subgrades for the project. Compression of these soils will occur due to the stresses resulting from dewatering during construction and the weight of the new junction box structures and the lined pipe trenches. Estimated settlements may exceed two to three inches using normal weight concrete for the junction boxes. Recommendations including the use of light weight concrete for construction of the junction box structures are included in the report. Light weight concrete is expected to have the same resistance to attack by organic compounds such as creosote as normal weight concrete.

Pipeline Support

The natural sands of Strata B and C encountered in the test borings are typically loose to very loose at and below the water table. These sands may exhibit characteristics of “running sands” when excavated. The contractor should be prepared to work with running sand conditions.

The Geotechnical Engineer should observe excavated pipe trench subgrades prior to any undercutting below design grades or placement of the liner materials. This is recommended to evaluate whether actual subgrade conditions are as anticipated based on our analysis.

Limited undercutting in the new storm drain line trenches may be recommended where soft or loose soils are encountered. This may be expected in areas where pipeline construction extends below the water table. Because the depth of undercut needed at any given location may vary, the Geotechnical Engineer should evaluate the actual undercut depths. The undercut materials may be replaced with crushed stone meeting the gradation requirements of VDOT No. 57 open-graded aggregate. Crushed concrete meeting this gradation may also be used. A layer of 10 oz. non-woven geotextile separation fabric or equivalent is recommended to be placed on the excavated subgrades prior to stone placement and over the stone prior to liner placement. We recommend evaluating undercut volumes by cross sectioning. Other methods of calculating volumes of undercut, such as counting trucks, are less accurate and generally result in additional expense.

Once the subgrade for the bottom of the trench is prepared, the trench liner materials may be placed. The storm drain pipes should be bedded according to manufacturer's specifications. Backfill over the top of the pipe should also be placed according to manufacturer's recommendations.

Our subsurface exploration revealed creosote treated wood to a maximum depth of 4 ft and 8 ft in Borings B-103 and B-104, respectively. Some existing structures may be present on the site and unknown quantities of wood may be buried within the footprint of the new storm drain alignment. Therefore, grading activities may encounter buried foundations and other associated debris. We recommend the complete removal of existing foundations and any wood materials from within the proposed storm drain alignment area. The contractor should remove existing foundations and/or debris in the proposed construction areas to at least 2 ft below the design subgrade level to expose the suitable subgrades of Strata B and C or replace the unsuitable material with compacted structural fill.

Coarse-grained structural fill is recommended for backfill of the pipe trenches. A layer of separation geotextile fabric should be placed between soil structural fill and the pipe bedding material.

Coarse grained compacted structural fill should consist of material classifying SC, SM, SP, SW, GC, GM, GP, or GW per ASTM D2487 and have a maximum compacted wet unit weight 110 pcf. Off-site borrow soils are anticipated to be used as compacted structural fill for the Storm Drain trenches.

The coarse-grained compacted structural fill should be placed in maximum eight-inch thick horizontal, loose lifts and should be compacted to at least 95 percent of maximum dry density per ASTM D698, Standard Proctor. The contractor should bench compacted structural fill subgrades steeper than 4H:1V to allow placement of horizontal lifts.

The last foot of compacted fill is to be a low permeability material meeting the requirements of a dense graded crusher run CR-6 aggregate. The gradation requirements of this material are similar to those of a VDOT No. 25 or 26 crusher run aggregate. We recommend that this material have a CBR Value of at least 20. The low permeability compacted structural fill should be placed in maximum eight-inch thick horizontal, loose lifts and should be compacted to at least 98 percent of maximum dry density per ASTM D698, Standard Proctor. The CR-6 material when placed as recommended is expected to provide a suitable subgrade for gantry crane travel.

Since the pipeline trench is lined and covered with a low permeability material, we have considered the trench as a long foundation for evaluating settlements. We have estimated soil contact pressures of between about 600 psf to 900 psf for the pipeline trench. Estimated settlements up to about 2 inches may occur after construction, before groundwater levels return to their normal levels. These settlements may vary depending on the duration of dewatering after construction.

Cast-in-Place Pipe Junction Boxes

The storm drain pipe junction boxes will be reinforced concrete structures. We recommend that the larger Junction Box Structures SD 1 and SD 2 be constructed using light weight concrete having a unit weight of not more than 120 pcf. We consider that the mat slab foundation of the pipe junction box can be supported on suitable coarse-grained soils of Strata A, B1, and C1. Where fine-grained soils of Strata B1 and C12 are encountered at the excavated subgrade, they should be undercut two feet. Undercut materials should be replaced with crushed stone or crushed concrete meeting the gradation requirements of VDOT No. 57 stone. A 10 oz. non-woven geotextile separation fabric or equivalent is recommended to be placed over the excavated subgrade prior to backfilling to design grades with crushed stone. These soils are expected to be suitable for the junction box soil contact pressures.
We have estimated soil contact pressures of about 900 psf and 550 psf for Junction Box Structures SD-1 and SD-2, respectively. These contact pressures are for construction in a dewatered excavation. The contact pressure reduces to about 215 psf for Box SD-1 with a ground water table at the ground surface. Box SD-2 could potentially be buoyant under like conditions. This assumes both boxes are dry.

Junction Box uplift resistance should be taken as the buoyant weight of the junction box concrete and the weight of a soil wedge surrounding the foundation as shown in Figure 3. Buoyant unit weights of the concrete and soil are 57.6 pcf and 47.6 pcf, respectively. We have estimated factors of safety for uplift of at least 1.7. A factor of safety of at least 1.5 should be used to design the structures against uplift.

Settlements of cast-in-place pipe junction boxes supported on suitable natural soils and on properly placed compacted aggregate fill are not expected to exceed about 1.5 inches after construction when they are cast with light weight concrete. As with the pipe trench settlements, these settlements may vary depending on the duration of dewatering after construction. Differential settlements between storm drain pipelines and cast-in-place junction boxes are not expected to exceed about half of this value. This evaluation considered a slight increase in the preconsolidation pressure of the upper compressible foundation soils due to the early effects of construction dewatering and the resulting increase of the effective stress to these soils.

A six-inch crushed stone or crushed concrete working platform should underlie the mat slabs of the pipe junction boxes. The crushed aggregate should meet the gradation requirements of VDOT No. 57 crushed stone. The contractor should compact the stone in place by at least two passes with suitable vibratory compaction equipment. A non-woven separation geotextile fabric such as a 10 oz. fabric or equivalent should be placed over the excavated subgrade prior to placement of the working platform stone.

Excavation Support

Based on the test boring data, soils to be excavated during project construction generally consist of existing Fill and Alluvium. These granular soils are considered as OSHA Type C soils for open cut excavations. OSHA guidelines indicate that a maximum slope angle of 1-1/2H:1V may be used for Type C soils.

Recommended soil parameters for design of a temporary braced excavation are provided below. These parameters have been estimated from the test boring and soil laboratory test data obtained in this study.

Strata	Depth (feet)	Classification	Total Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
A/B	0-10	SC/SP/SM	115	32	-
C1	10-15	SP/SM	105	30	-
C2	15-40	CH/CL	100	6	200

CONSTRUCTION CONSIDERATIONS

Earthwork

We expect the subgrade soils in sections of the pipeline excavations to be wet and easily disturbed. The contractor may need crushed stone and stabilization geotextile working platforms to provide a base on which to place compacted liner materials and/or structural fill. The Geotechnical Engineer can make recommendations for working platforms in the field, based on observation of subgrade conditions.

Construction Dewatering

We anticipate that the contractor will encounter groundwater during excavation for the proposed storm drain pipeline and junction box structures. Pumping from a well-point system will likely be necessary to control the groundwater levels in the excavations. A well-point system consists of multiple well-points placed around the excavation, all connected to a universal header, which is then attached to a pump. Project specifications should make the contractor responsible for dewatering methods. Collection and disposal of pumped groundwater should follow the project specifications.

Excavation Support

We anticipate that storm drain junction box excavations may require sheeting for excavation support. Where excavation sheeting is used, the excavation contractor should prepare drawings indicating details of the excavation sheeting. A licensed Professional Engineer should prepare the drawings and should submit the drawings to the Structural Engineer and to our office for review.

We recommend that the contract require the excavation sheeting and shoring contractor to furnish bodily injury and property damage liability insurance to adequately protect the Owner and consulting engineers on the project from claims arising from the work. The builder's risk policy should also name the Engineer and Schnabel Engineering Consultants, Inc. as co-insured for claims arising from construction.
A specialty contractor, who has had at least 5 years experience in performance of the specialized work, should perform the excavation sheeting, including preparation of the plans.

Concrete Pipe Junction Boxes

The contractor should exercise care during excavation for pipe junction box floor slabs so that as little disturbance as possible occurs at the slab level. The contractor should carefully clean loose or soft soils from the bottom of the excavation before placing concrete.

Base subgrades needing undercut should be backfilled to the original design subgrade elevation with an open-graded crushed stone or crushed concrete such meeting the gradation requirements of VDOT No. 57 aggregate. Crushed stone should extend at least six inches laterally beyond the base in all directions. A non-woven separation geotextile fabric such as a 10 oz. fabric or equivalent should be placed over the excavated subgrade prior to backfilling to design grade.

Engineering Services During Construction

The engineering recommendations provided in this report are based on the information obtained from the subsurface exploration and laboratory testing. However, conditions on the site may vary between the discrete locations observed at the time of our subsurface exploration. The nature and extent of variations between borings may not become evident until during construction.

To account for this variability, we should provide professional observation and testing of actual subsurface conditions revealed during construction as an extension of our engineering services. These services will also help in evaluating the contractor's conformance with the plans and specifications. Because of our unique position to understand the intent of the geotechnical engineering recommendations, retaining Schnabel for these services will allow us to provide consistent service throughout the project construction.

General Specification Recommendations

An allowance should be established to account for possible additional costs that may be required to construct earthwork and foundations as recommended in this report. Additional costs may be incurred for a variety of reasons including variation of soil between borings, greater than anticipated unsuitable soils, need for borrow fill material, obstructions, temporary dewatering, etc.

We recommend that the construction contract include an allowance for undercutting soft or loose, near-surface soils, and replacement with compacted structural fill. Add/deduct unit prices should also be established in the contract so adjustments can be made for the actual volume of materials handled.

The project specifications should indicate the contractor's responsibility for providing adequate site drainage during construction. Inadequate drainage will most likely lead to disturbance of soils by construction traffic and increased volume of undercut.

This report may be made available to prospective bidders for informational purposes. We recommend that the project specifications contain the following statement:

> *Schnabel has prepared this geotechnical engineering report for this project. This report is for informational purposes only and is not part of the contract documents. The opinions expressed represent the Geotechnical Engineer's interpretation of the subsurface conditions, tests, and the results of analyses conducted. Should the data contained in this report not be adequate for the Contractor's purposes, the Contractor may make, before bidding, independent exploration, tests and analyses. This report may be examined by bidders at the office of the Owner, or copies may be obtained from the Owner at nominal charge.*

The contract documents should include the boring data provided in Appendix B.

Additional data and reports prepared by others that could have an impact upon the contractor's bid should also be made available to prospective bidders for informational purposes.

LIMITATIONS

We based the analyses and recommendations submitted in this report on the information revealed by our exploration. We attempted to provide for normal contingencies, but the possibility remains that unexpected conditions may be encountered during construction.

We prepared this report to aid in the evaluation of this site and to assist in the design of the project. We intend it for use concerning this specific project. We based our recommendations on information on the site and proposed construction as described in this report. Substantial changes in loads, locations, or grades should be brought to our attention so we can modify our recommendations as needed. We would appreciate an opportunity to review the plans and specifications as they pertain to the recommendations contained in this report, and to submit our comments to you based on this review.

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project. No other representation, express or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or any other instrument of service.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

Sincerely,

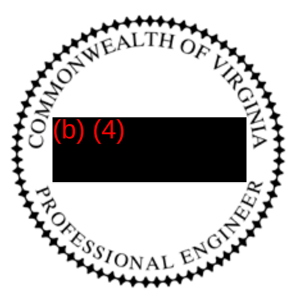
SCHNABEL ENGINEERING, LLC

(b) (4) [redacted]

(b) (4) [redacted]
Senior Staff Engineer

(b) (4) [redacted]

(b) (4) [redacted]
Principal



FJR:GTS:adh

Figures
Appendix A: Soil Laboratory Test Data
Appendix B: Subsurface Exploration Data

Distribution:

EA Engineering, Science and Technology, Inc.
Attn: Heather Guinivan

EA Engineering, Science and Technology, Inc.
Attn: Mark Gutberlet, PE

EA Engineering, Science and Technology, Inc.
Attn: Pete Pellissier, PE

FIGURES

Source: ESRI Online Premium Service (©2011 BING)
Projection: WGS 1984 Web Mercator Auxiliary Sphere

250 125 0 250 Feet

Scale: 1:3,041

Schnabel ENGINEERING

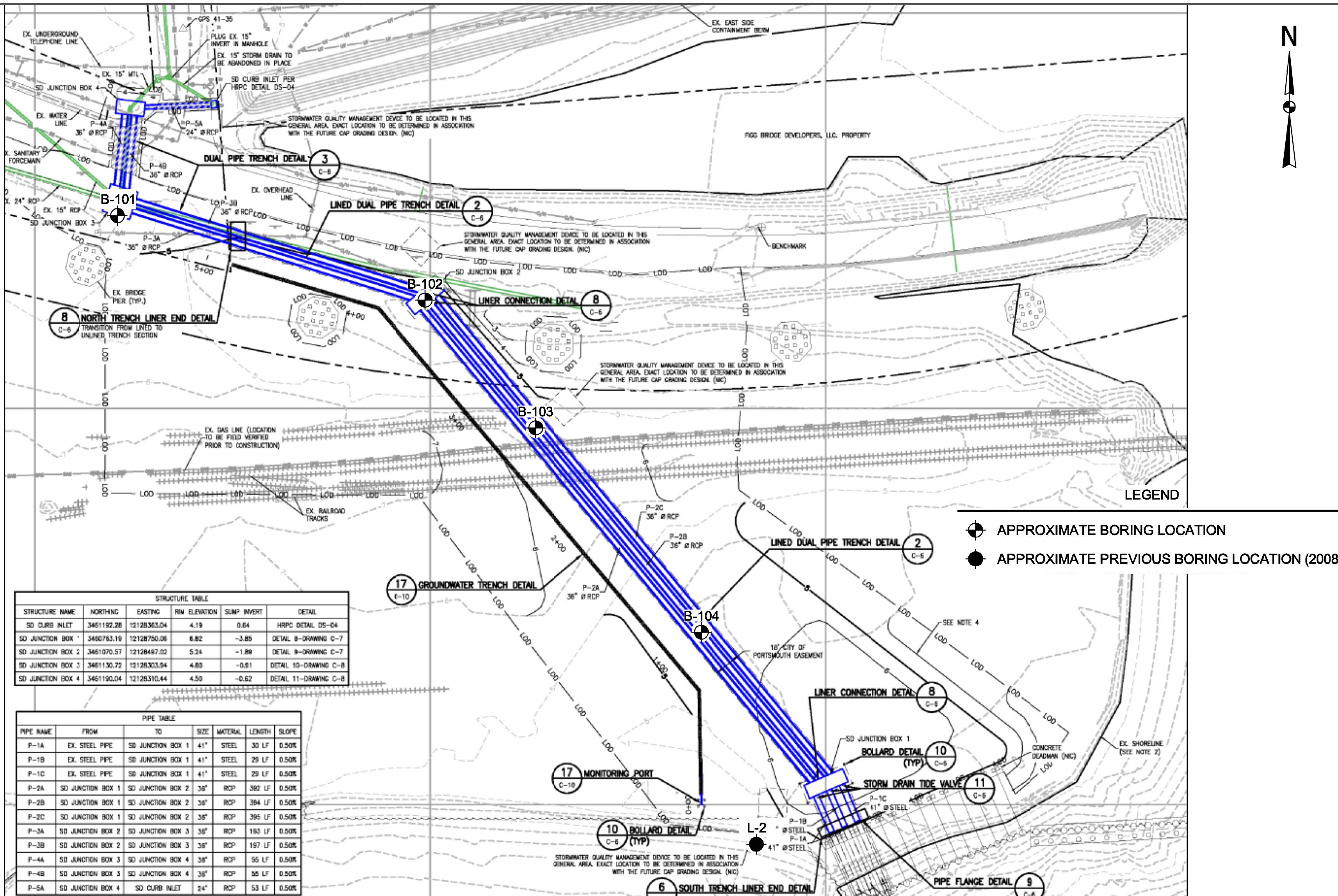
AWI EASDR
ELM AVENUE AT VENEER ROAD
PORTSMOUTH, VIRGINIA
PROJECT NO. 10233031.05

SITE VICINITY MAP

FIGURE 1

jhalligan

6/21/2012 B:\GIS_Data\2010\10233031 - AWI Storm Drain Relocation\Projects\Figure 1 - Site Vicinity Map.mxd



STRUCTURE TABLE					
STRUCTURE NAME	NORTHING	EASTING	RIM ELEVATION	SUMP INVERT	DETAIL
SD CURB INLET	3461192.28	12128363.04	4.19	0.64	HRPC DETAIL DS-04
SD JUNCTION BOX 1	3460763.19	12128750.06	6.82	-3.85	DETAIL 8-DRAWING C-7
SD JUNCTION BOX 2	3461070.57	12128497.02	5.24	-1.89	DETAIL 9-DRAWING C-7
SD JUNCTION BOX 3	3461130.72	12128303.94	4.80	-0.61	DETAIL 10-DRAWING C-8
SD JUNCTION BOX 4	3461190.04	12128310.44	4.50	-0.62	DETAIL 11-DRAWING C-8

PIPE TABLE						
PIPE NAME	FROM	TO	SIZE	MATERIAL	LENGTH	SLOPE
P-1A	EX. STEEL PIPE	SD JUNCTION BOX 1	41"	STEEL	30 LF	0.50%
P-1B	EX. STEEL PIPE	SD JUNCTION BOX 1	41"	STEEL	29 LF	0.50%
P-1C	EX. STEEL PIPE	SD JUNCTION BOX 1	41"	STEEL	29 LF	0.50%
P-2A	SD JUNCTION BOX 1	SD JUNCTION BOX 2	36"	RCP	392 LF	0.50%
P-2B	SD JUNCTION BOX 1	SD JUNCTION BOX 2	36"	RCP	394 LF	0.50%
P-2C	SD JUNCTION BOX 1	SD JUNCTION BOX 2	36"	RCP	395 LF	0.50%
P-3A	SD JUNCTION BOX 2	SD JUNCTION BOX 3	36"	RCP	193 LF	0.50%
P-3B	SD JUNCTION BOX 2	SD JUNCTION BOX 3	36"	RCP	197 LF	0.50%
P-4A	SD JUNCTION BOX 3	SD JUNCTION BOX 4	36"	RCP	55 LF	0.50%
P-4B	SD JUNCTION BOX 3	SD JUNCTION BOX 4	36"	RCP	55 LF	0.50%
P-5A	SD JUNCTION BOX 4	SD CURB INLET	24"	RCP	53 LF	0.50%

Base Plan Provided by EA ENGINEERING, SCIENCE AND TECHNOLOGY dated MARCH 2012.

SCALE: 1"=60'
0 60' 120'

Schnabel ENGINEERING

ELM AVENUE STORM DRAIN RELOCATION (EASDR)
ATLANTIC WOOD INDUSTRIES
PORTSMOUTH, VIRGINIA

Figure Name:	Done:	Figure Number:
BORING LOCATION PLAN	F. ROMANO	2
Project Number: 10233031.05	Reviewed: G. SEESE	Date: JUNE 2012

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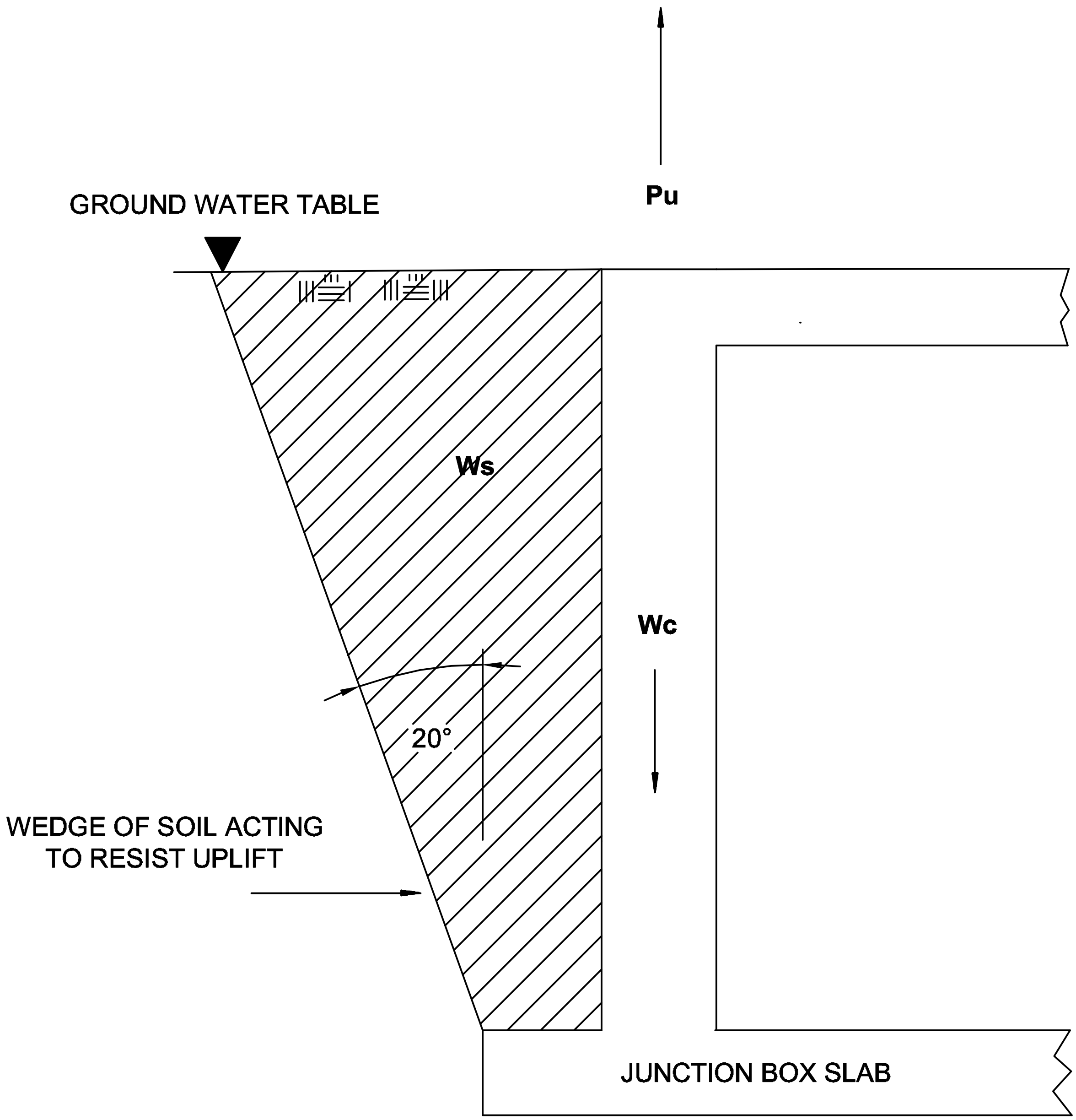
UPLIFT RESISTANCE

NOT TO SCALE



LEGEND

Wc = BUOYANT WEIGHT OF CONCRETE = 57.6 pcf

Ws = BUOYANT WEIGHT OF SOIL = 47.6 pcf

Pu = UPLIFT LOAD

Schnabel ENGINEERING	AWI - ELM AVENUE STORM DRAIN REPLACEMENT, PORTSMOUTH, VIRGINIA	Figure Name: UPLIFT RESISTANCE	Done: F. ROMANO	Figure Number: 3
		Project Number: 10233031.05	Reviewed: G. SEESE	Date: JUNE 2012

APPENDIX A

SOIL LABORATORY TEST DATA

Summary of Soil Laboratory Tests (2 sheets)
Gradation Curves (4 sheets)
Atterberg Limits (1 sheet)
Consolidation Curves (2 sheets)

Summary Of Laboratory Tests

Boring No.	Sample Depth ft / Elevation ft	Sample Type	Description of Soil Specimen	Stratum	Wet Natural Density (pcf)	Natural Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	% Passing No. 40 Sieve	% Passing No. 200 Sieve	Specific Gravity
B-101	6.0 - 8.0 -1.2 - -3.2	Jar	POORLY GRADED SAND (SP), fine to medium grained sand, gray	C1	--	25.6	--	--	--	88.5	4.2	--
B-101	9.0 - 10.0 -4.2 - -5.2	Jar	FAT CLAY WITH SAND (CH), contains organics, dark gray	C2	--	59.3	69	28	41	96.9	74.5	--
B-102	8.0 - 10.0 -4.8 - -6.8	Jar	POORLY GRADED SAND WITH SILT (SP-SM), fine to medium grained sand, gray	C1	--	22.2	--	--	--	93.3	8.3	--
B-102	15.0 - 17.0 -11.8 - -13.8	Tube	FAT CLAY (CH), contains sand - gray	C2	97.5	71.6	61	29	32	99.8	98.9	2.71
B-102	23.0 - 25.0 -19.8 - -21.8	Jar	FAT CLAY (CH), contains organics, dark gray	C2	--	67.9	76	31	45	100.0	97.7	--
B-103	13.0 - 15.0 -5.5 - -7.5	Jar	POORLY GRADED SAND WITH SILT (SP-SM), fine to medium grained sand, contains shell fragments and organics, dark gray	C1	--	30.6	--	--	--	95.2	11.8	--
B-103	18.0 - 20.0 -10.5 - -12.5	Jar	FAT CLAY (CH), contains shell fragments, dark gray	C2	--	77.1	76	32	44	98.1	92.1	--

Notes:
1. Soil tests in general accordance with ASTM standards.
2. Soil classifications are in general accordance with ASTM D2487(as applicable), based on testing indicated and visual classification.
3. Key to abbreviations: NP=Non-Plastic; -- indicates no test performed

Schnabel ENGINEERING

Project: AWI Elm Avenue Storm Drain Relocation
Elm Avenue and Veneer Road
Portsmouth, VA

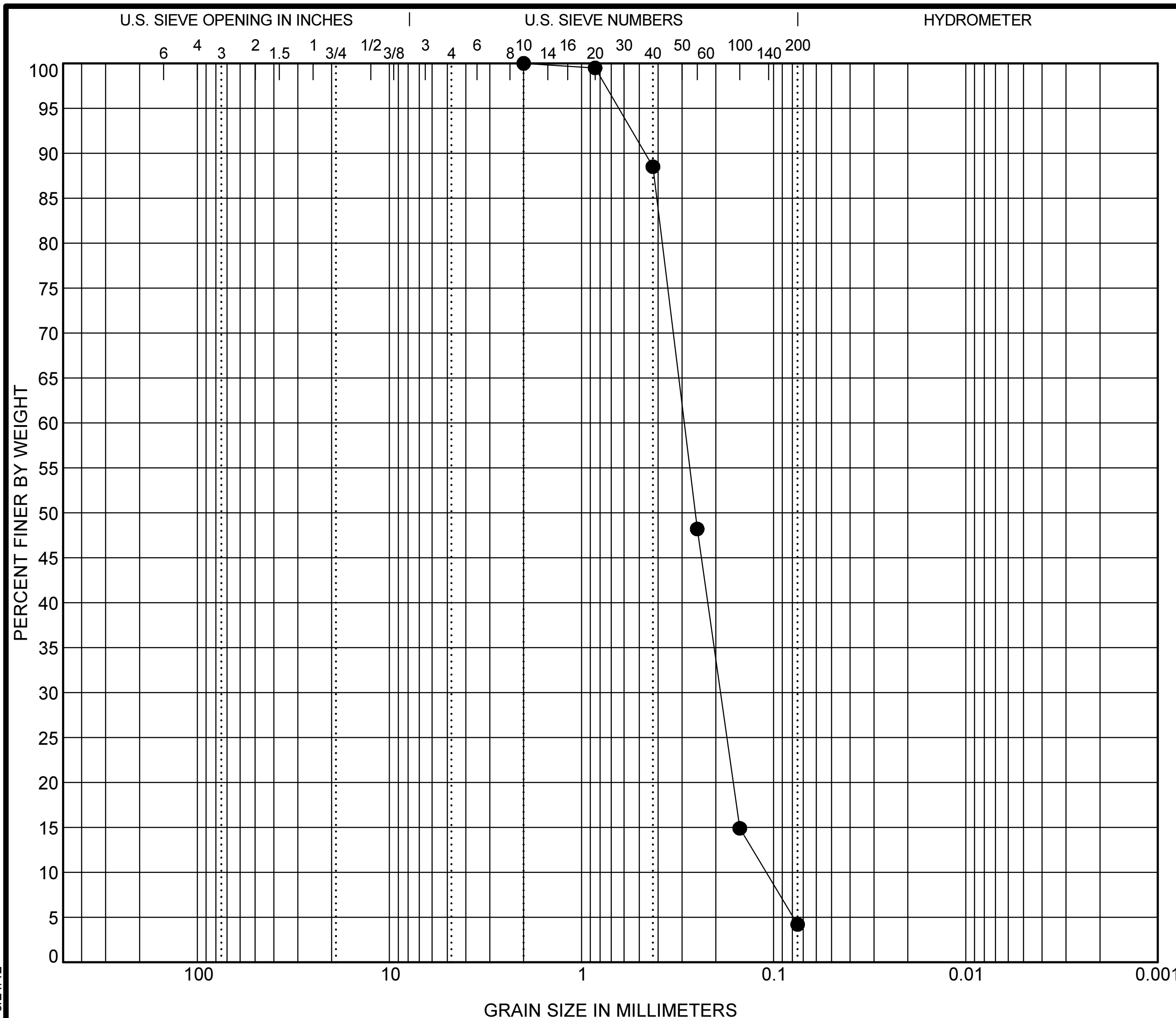
Summary Of Laboratory Tests

Boring No.	Sample Depth ft / Elevation ft	Sample Type	Description of Soil Specimen	Stratum	Wet Natural Density (pcf)	Natural Moisture (%)	Liquid Limit	Plastic Limit	Plasticity Index	% Passing No. 40 Sieve	% Passing No. 200 Sieve	Specific Gravity
B-104	13.0 - 15.0 -6.4 - -8.4	Jar	POORLY GRADED SAND (SP), fine to medium grained sand, contains shell fragments, gray	C1	--	21.3	--	--	--	78.7	4.5	--
B-104	18.0 - 20.0 -11.4 - -13.4	Jar	FAT CLAY (CH), dark gray	C2	--	70.5	74	29	45	99.4	90.9	--
B-104	30.0 - 32.0 -23.4 - -25.4	Tube	FAT CLAY (CH), contains sand - gray	C2	100.0	66.9	66	31	35	99.9	99.2	2.70

Notes:
1. Soil tests in general accordance with ASTM standards.
2. Soil classifications are in general accordance with ASTM D2487(as applicable), based on testing indicated and visual classification.
3. Key to abbreviations: NP=Non-Plastic; -- indicates no test performed

Schnabel ENGINEERING

Project: AWI Elm Avenue Storm Drain Relocation
Elm Avenue and Veneer Road
Portsmouth, VA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen		Sample Description	LL	PL	PI		
●	B-101	6.0 ft	**POORLY GRADED SAND (SP), fine to medium grained sand, gray**	**NP**	**NP**	**NP**		

Test Method	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
ASTM D422	**2**	**0.292**	**0.189**	**0.109**			**4.2**	

Percent Finer

Sieve Size	No. 200	No. 100	No. 60	No. 40	No. 20	No. 10
% Finer	04.2	14.9	48.2	88.5	99.5	100.0

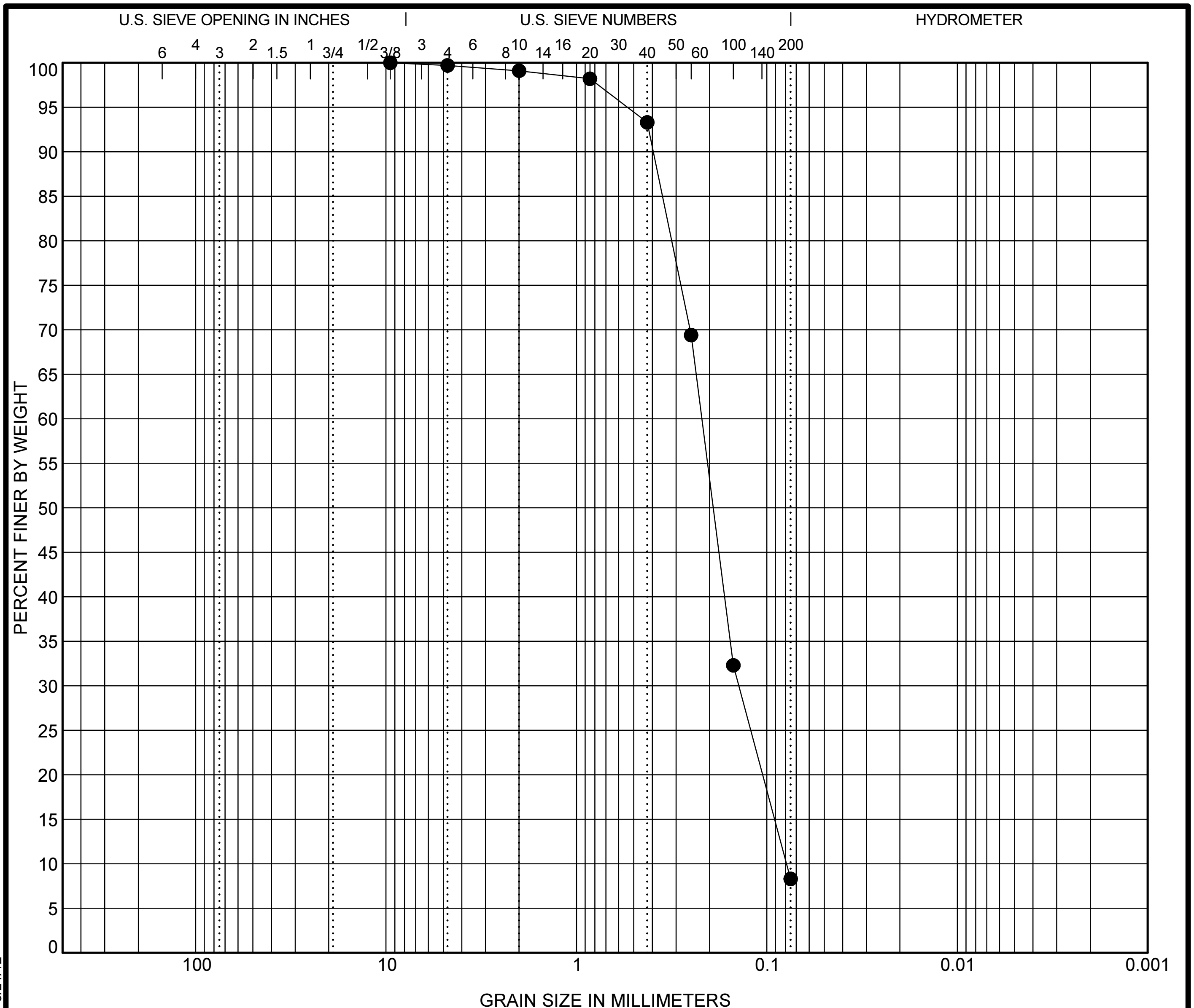
Tested By	Tested Date	Reviewed By	Calc By
KR	5/7/12		

Schnabel ENGINEERING

GRADATION CURVE

Project: AWI Elm Avenue Storm Drain Relocation
Elm Avenue and Veneer Road
Portsmouth, VA

Contract: 10233031.05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen		Sample Description	LL	PL	PI		
●	B-102	8.0 ft	**POORLY GRADED SAND WITH SILT (SP-SM), fine to medium grained sand, gray**	**NP**	**NP**	**NP**		

Test Method	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
ASTM D422	**9.5**	**0.22**	**0.14**	**0.079**	**0.3**	**91.4**	**8.3**	

Percent Finer

Sieve Size	No. 200	No. 100	No. 60	No. 40	No. 20	No. 10	No. 4	3/8
% Finer	08.3	32.3	69.4	93.3	98.2	99.1	99.7	100.0

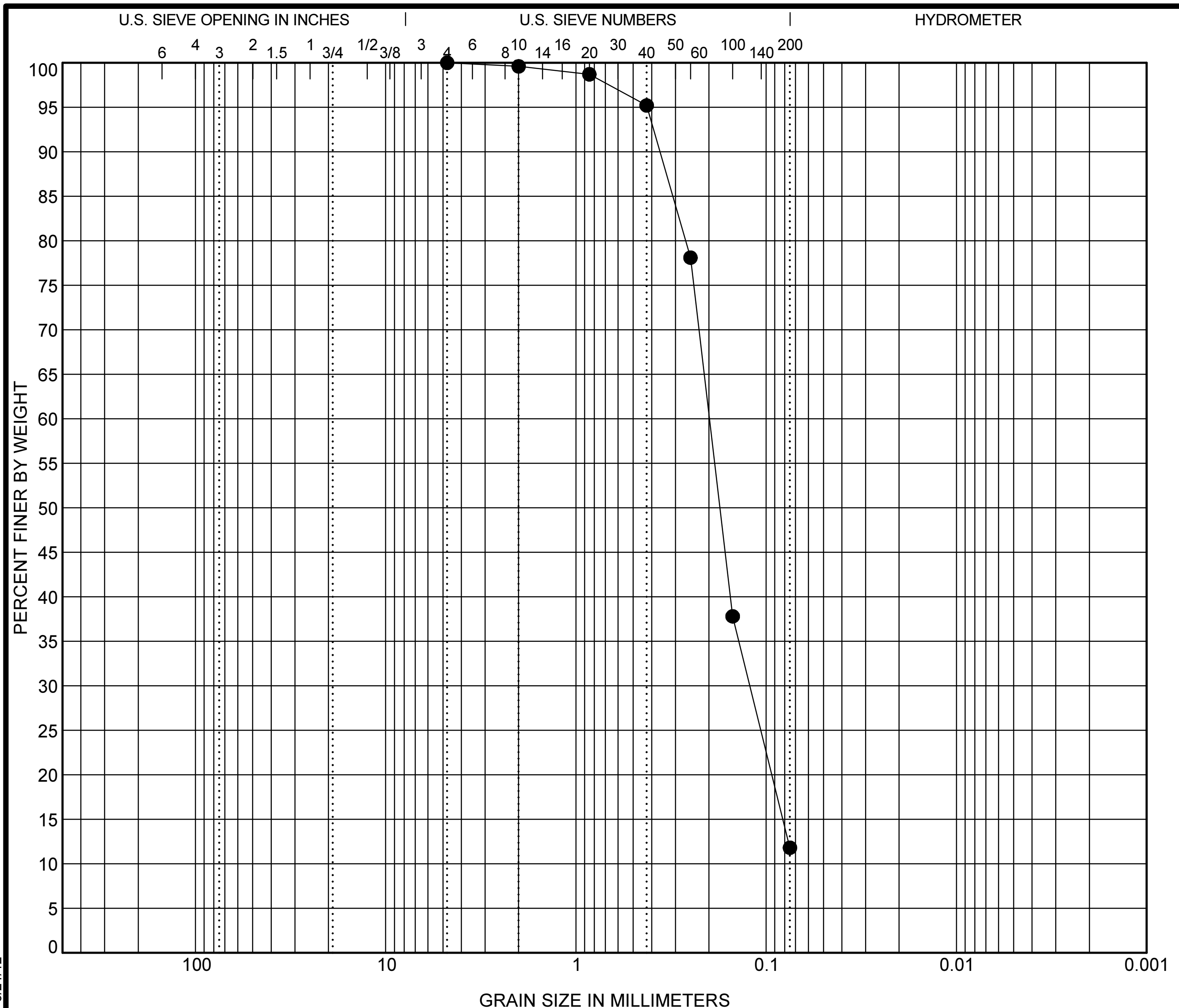
Tested By	Tested Date	Reviewed By	Calc By
KR	5/7/12		

Schnabel ENGINEERING

GRADATION CURVE

Project: AWI Elm Avenue Storm Drain Relocation
Elm Avenue and Veneer Road
Portsmouth, VA

Contract: 10233031.05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen		Sample Description	LL	PL	PI		
●	B-103	13.0 ft	**POORLY GRADED SAND WITH SILT (SP-SM), fine to medium grained sand, contains shell fragments and organics, dark gray**	**NP**	**NP**	**NP**		

Test Method	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
ASTM D422	**4.75**	**0.199**	**0.122**		**0.0**	**88.2**	**11.8**	

Percent Finer

Sieve Size	No. 200	No. 100	No. 60	No. 40	No. 20	No. 10	No. 4
% Finer	11.8	37.8	78.1	95.2	98.7	99.6	100.0

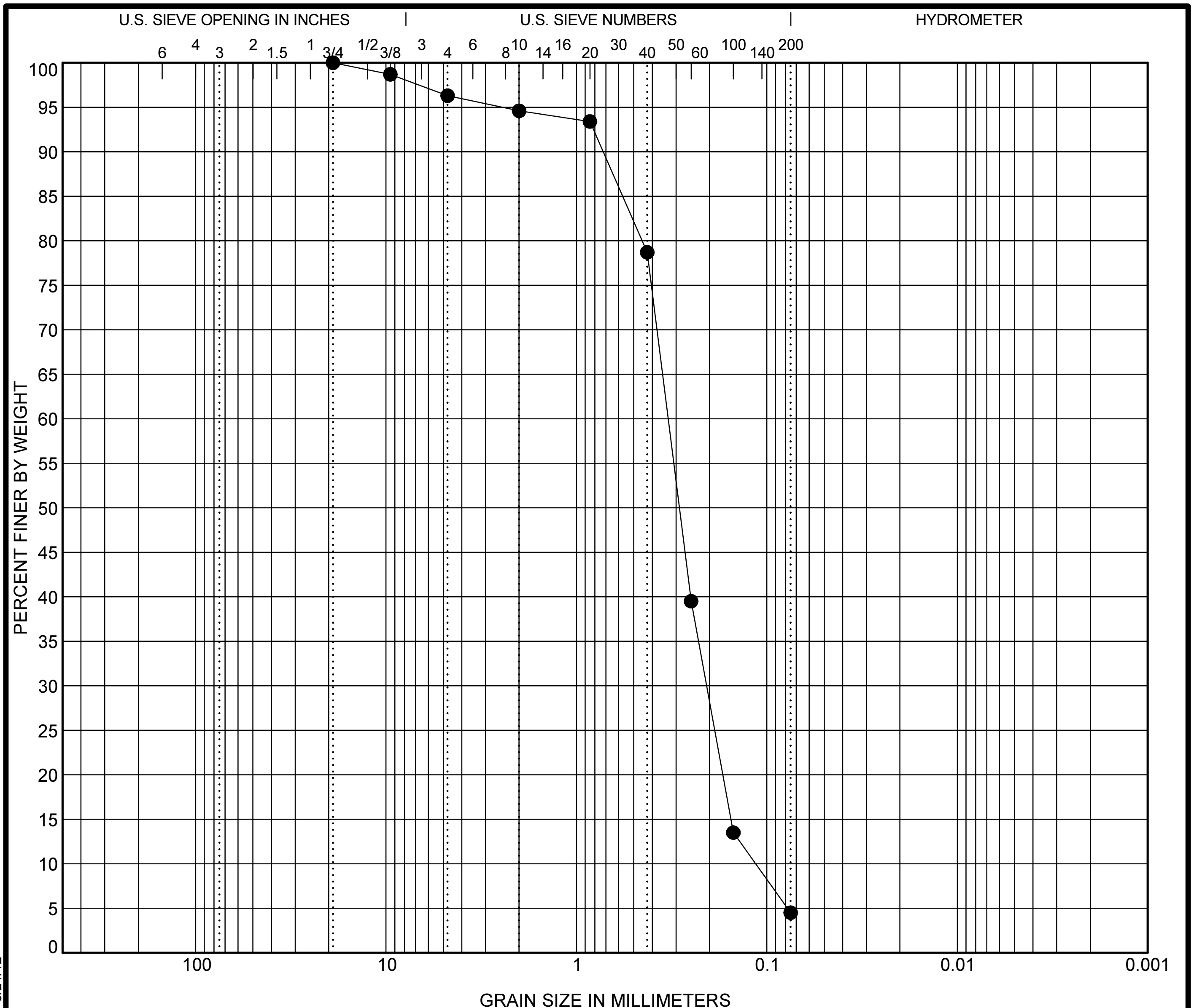
Tested By	Tested Date	Reviewed By	Calc By
KR	5/7/12		

Schnabel ENGINEERING

GRADATION CURVE

Project: AWI Elm Avenue Storm Drain Relocation
Elm Avenue and Veneer Road
Portsmouth, VA

Contract: 10233031.05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen		Sample Description	LL	PL	PI		
●	B-104	13.0 ft	**POORLY GRADED SAND (SP), fine to medium grained sand, contains shell fragments, gray**	**NP**	**NP**	**NP**		

Test Method	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
ASTM D422	**19**	**0.33**	**0.207**	**0.115**	**3.7**	**91.8**	**4.5**	

Percent Finer

Sieve Size	No. 200	No. 100	No. 60	No. 40	No. 20	No. 10	No. 4	3/8	3/4
% Finer	04.5	13.5	39.5	78.7	93.4	94.6	96.3	98.7	100.0

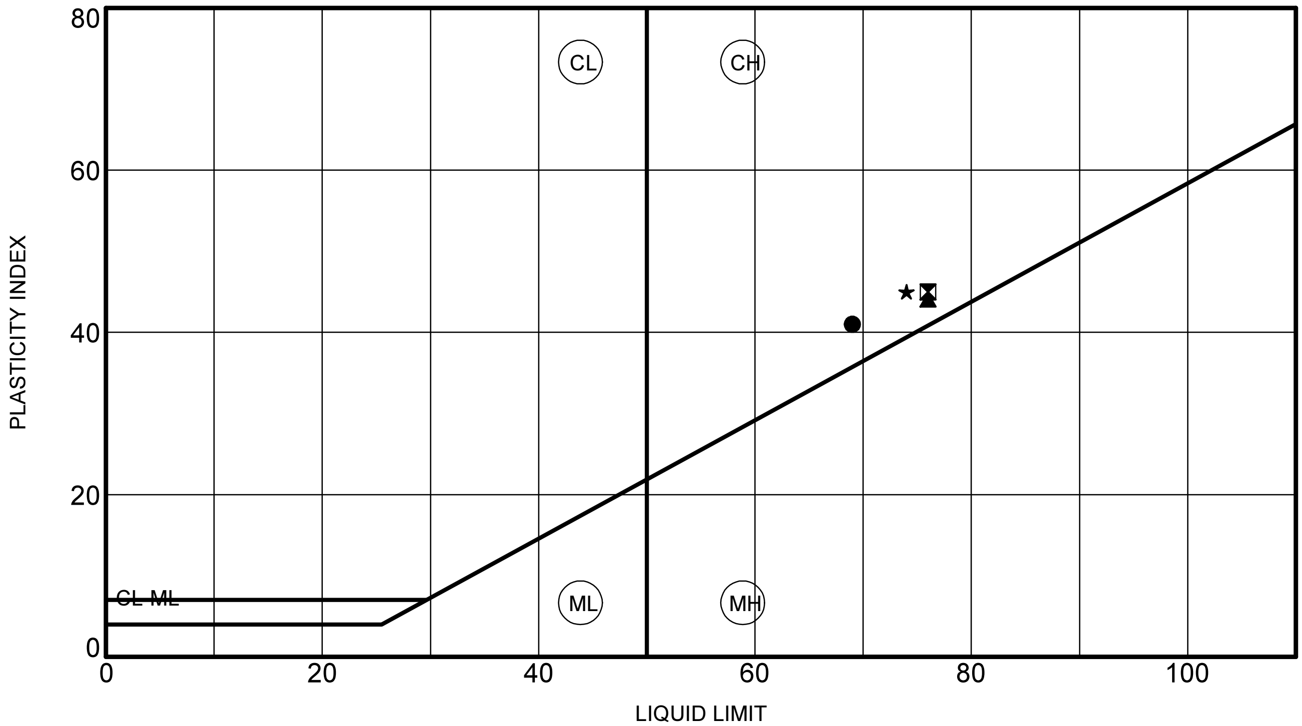
Tested By	Tested Date	Reviewed By	Calc By
KR	5/7/12		

Schnabel ENGINEERING

GRADATION CURVE

Project: AWI Elm Avenue Storm Drain Relocation
Elm Avenue and Veneer Road
Portsmouth, VA

Contract: 10233031.05



PLOTTED DATA REPRESENTS SOIL PASSING NO. 40 SIEVE

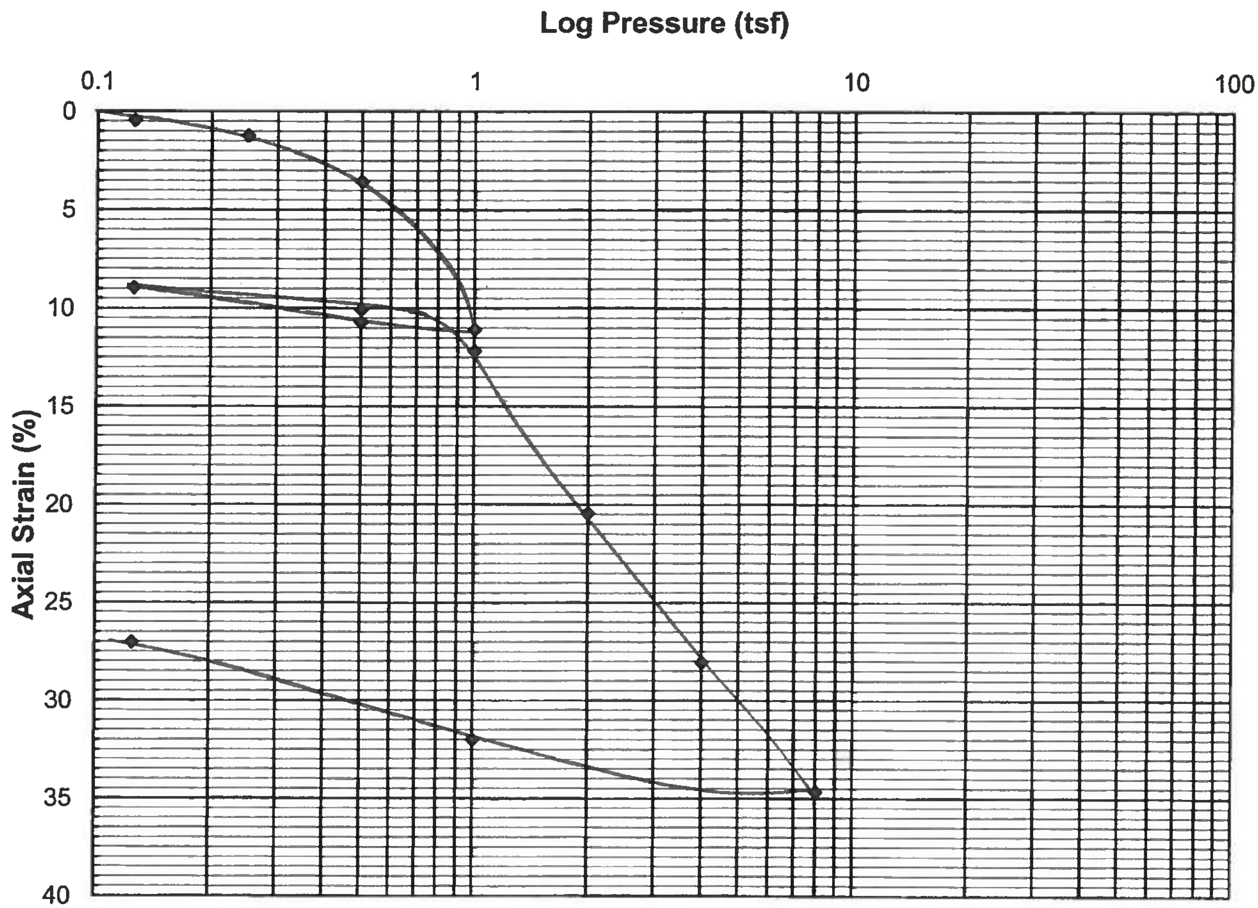
	Specimen		LL	PL	PI	Fines	Description
●	**B-101**	**9.0 ft**	**69**	**28**	**41**	**75**	**FAT CLAY WITH SAND (CH), contains organics, dark gray**
⧗	**B-102**	**23.0 ft**	**76**	**31**	**45**	**98**	**FAT CLAY (CH), contains organics, dark gray**
▲	**B-103**	**18.0 ft**	**76**	**32**	**44**	**92**	**FAT CLAY (CH), contains shell fragments, dark gray**
★	**B-104**	**18.0 ft**	**74**	**29**	**45**	**91**	**FAT CLAY (CH), dark gray**

Schnabel ENGINEERING

ATTERBERG LIMITS

Project: AWI Elm Avenue Storm Drain Relocation
Elm Avenue and Veneer Road
Portsmouth, VA

Contract: 10233031.05



Probable Preconsolidation Pressure (Pp), tsf:	*0.58*
Recompression Ratio (Cεr):	*0.020*
Compression Ratio (Cεc):	*0.284*

Type of Specimen: *Tube Sample*

Description: *FAT CLAY (CH), contains sand - gray*

LL:	*61*	**PI:**	*32*	**Gs:**	*2.71*	**P_o' (tsf):**	*0.39*
% < No. 200:	*98.9*	**Test Method:**	*ASTM D2435 Method A*				

Test Condition: *Inundated @ 0.05 tsf*

	Initial	Final
Water Content, %	76.4	46.1
Void Ratio	2.07	1.24
Saturation, %	100	100
Dry Unit Weight, pcf	55.1	75.6

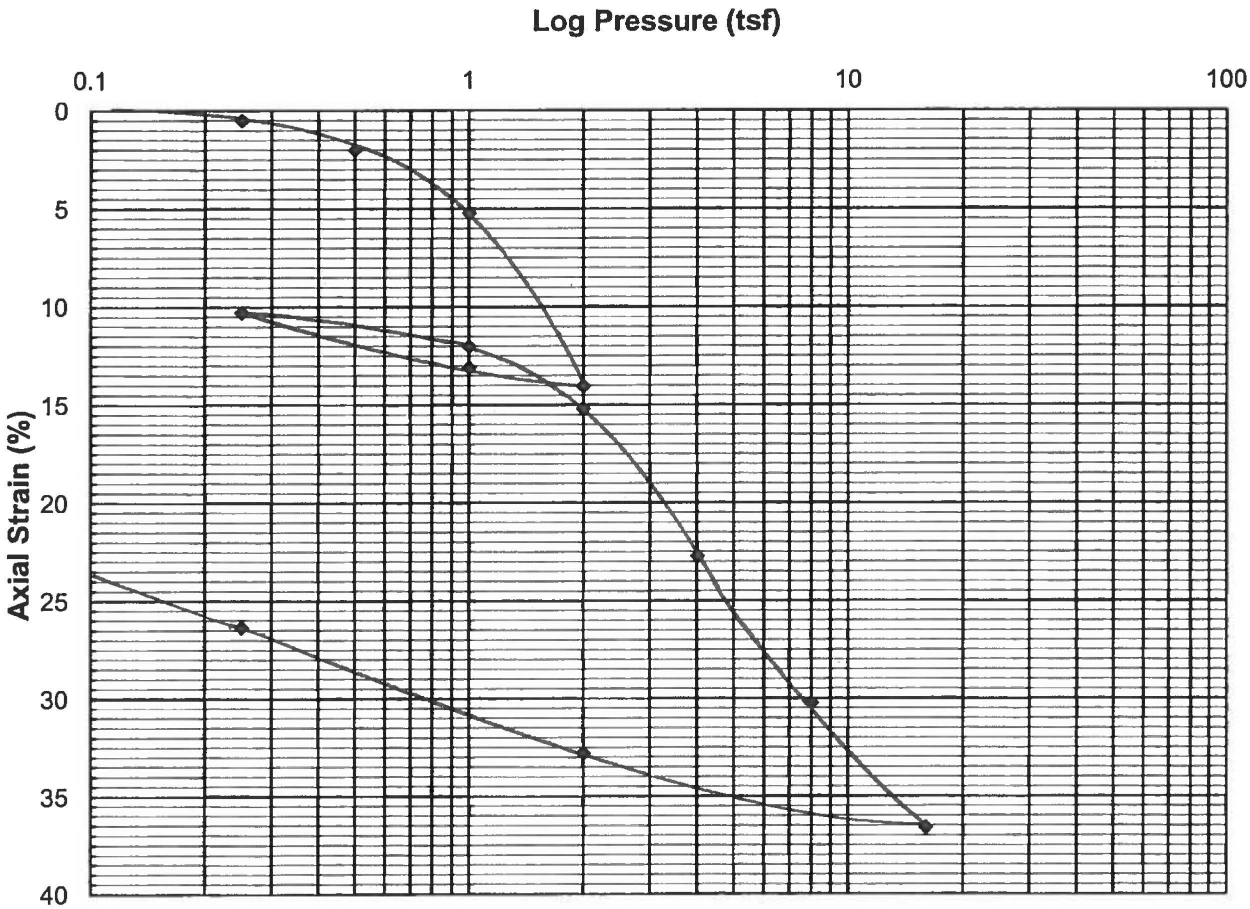
Remarks:

Average Water Content of Trimmings, %: *73.8*

Schnabel ENGINEERING

Project:	AWI Elm Avenue Storm Drain Relocation (EASDR)		
Location	*Portsmouth, VA*		
Boring:	*B-102*	**Schnabel No.:**	*10233031.05*
Depth:	*15-17 ft.*	**Elevation:**	*-12 to -14 ft.*
Date:	*5/30/2012*	**Reviewed by:**	*CJS*

Consolidation Test Report



Probable Preconsolidation Pressure (Pp), tsf: *0.96*		**Recompression Ratio (Cεr):**	*0.030*
Type of Specimen: *Tube Sample*		**Compression Ratio (Cεc):**	*0.274*
Description: *FAT CLAY (CH), contains sand - gray*		**Initial**	**Final**
	Water Content, %	*70.4*	*43.4*
LL: *66* **PI:** *35* **Gs:** *2.70* **P$_o$' (tsf):** *0.79*	**Void Ratio**	*1.89*	*1.13*
% < No. 200: *99.2* **Test Method:** *ASTM D2435 Method A*	**Saturation, %**	*100*	*100*
Test Condition: *Inundated @ 0.05 tsf*	**Dry Unit Weight, pcf**	*58.3*	*79.2*

Remarks: *1/8 tsf pressure applied to arrest swell upon inundation*

Average Water Content of Trimmings, %: *65.3*

Schnabel ENGINEERING

Project:	AWI Elm Avenue Storm Drain Relocation (EASDR)		
Location	*Portsmouth, VA*		
Boring:	*B-104*	**Schnabel No.:**	*10233031.05*
Depth:	*30-32 ft.*	**Elevation:**	*-23 to -25 ft.*
Date:	*5/30/2012*	**Reviewed by:**	*CJS*

Consolidation Test Report

APPENDIX B

SUBSURFACE EXPLORATION DATA

Subsurface Exploration Procedures
General Notes for Subsurface Exploration Logs
Identification of Soil
Boring Logs, B-101 through B-104, L-2

SUBSURFACE EXPLORATION PROCEDURES

Boring Procedures

Drillers advanced the borings using mud rotary drilling. With mud rotary drilling techniques, driller's mud is used to maintain an open bore hole. The hole is advanced by using a nominal 3-inch O.D. tri-cone roller bit. At the designated depth, drillers remove the roller bit and perform the Standard Penetration Test. Water level data indicated on the logs may not be indicative of actual groundwater levels because of the presence of drilling fluid in the borehole. The logs indicate water level data.

Standard Penetration Test Results

The numbers in the Sampling Data column of the boring logs represent Standard Penetration Test (SPT) results. Each number represents the blows needed to drive a two-inch O.D., 1⅜ inch I.D. split-spoon sampler six inches, using a 140-pound hammer falling 30 inches. The sampler is typically driven a total of 18 or 24 inches. The first six inch interval usually represents a seating interval. The total of the number of blows for the second and third six-inch intervals is the SPT "N value." When the blow count reaches 100 before the full driving distance, we determine the SPT N value based on extrapolation of the blows recorded. The SPT is conducted according to ASTM D1586.

Soil Classification Criteria

The group symbols on the logs represent the Unified Soil Classification System Group Symbols (ASTM D2487) based on visual observation and limited laboratory testing of the samples. Criteria for visual identification of soil samples are included in this appendix. Some variation may be expected between samples visually classified and samples classified in the laboratory.

Pocket Penetrometer Results

The values following "PP= " in the Sampling Data column of the logs represent pocket penetrometer readings. Pocket penetrometer readings provide an estimate of the unconfined compressive strength of fine-grained soils.

Water Observation Wells

Our drilling subcontractor installed temporary water observation wells in Borings B-101, B-102, and B-104 by inserting a hand-slotted, 1¼-inch PVC pipe in each of these borings. Each pipe was capped, and the area surrounding the pipe was backfilled with cuttings from the boring. The pipes were later removed and the holes were backfilled with grout.

Boring Locations and Elevations

Personnel from Baldwin and Gregg, Ltd. performed a boring stakeout and an elevation survey of the boring locations. Figure 2 shows the approximate boring locations. Project planning should consider these locations and elevations no more accurate than the methods and plans used to obtain them.

GENERAL NOTES FOR SUBSURFACE EXPLORATION LOGS

1. Numbers in sampling data column next to Standard Penetration Test (SPT) symbols indicate blows required to drive a 2-inch O.D., 1⅜-inch I.D. sampling spoon 6 inches using a 140 pound hammer falling 30 inches. The Standard Penetration Test (SPT) N value is the number of blows required to drive the sampler 12 inches, after a 6-inch seating interval. The Standard Penetration Test is performed in general accordance with ASTM D1586.
2. Visual classification of soil is in accordance with terminology set forth in “Identification of Soil.” The ASTM D2487 group symbols (e.g., CL) shown in the classification column are based on visual observations.
3. Estimated water levels indicated on the logs are only estimates from available data and may vary with precipitation, porosity of the soil, site topography, and other factors.
4. Refusal at the surface of rock, boulder, or other obstruction is defined as an SPT resistance of 100 blows for 2 inches or less of penetration.
5. The logs and related information depict subsurface conditions only at the specific locations and at the particular time when drilled or excavated. Soil conditions at other locations may differ from conditions occurring at these locations. Also, the passage of time may result in a change in the subsurface soil and water level conditions at the subsurface exploration location.
6. The stratification lines represent the approximate boundary between soil and rock types as obtained from the subsurface exploration. Some variation may also be expected vertically between samples taken. The soil profile, water level observations and penetration resistances presented on these logs have been made with reasonable care and accuracy and must be considered only an approximate representation of subsurface conditions to be encountered at the particular location.
7. Key to symbols and abbreviations:

S-1, SPT	Sample No., Standard Penetration Test
5+10+1	Number of blows in each 6-inch increment
UD-1, UNDIST	Sample No., 2” or 3” Undisturbed Tube Sample
Rec=24”, 100%	Recovery in inches, Percent Recovery
C-1, CORE	Core No., Rock Core
Run = 5.0 ft	Run length in feet
REC = 60", 100%	Recovery in inches, Percent Recovery
RQD = 60", 100%	RQD in inches, Percent RQD
MC	Moisture Content
PP	Pocket Penetrometer Reading (tsf)
FD	Flame Ionization Detector Reading (ppm)
PD	Photoionization Detector Reading (ppm)
GP	Geostick Penetration Reading (inches)
LL	Liquid Limit
PL	Plastic Limit
TPH	Total Petroleum Hydrocarbons

IDENTIFICATION OF SOIL

I. DEFINITION OF SOIL GROUP NAMES (ASTM D2487)

			SYMBOL	GROUP NAME
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels – More than 50% of coarse fraction retained on No. 4 sieve Coarse, ¾” to 3” Fine, No. 4 to ¾”	Clean Gravels Less than 5% fines	GW	WELL GRADED GRAVEL
			GP	POORLY GRADED GRAVEL
		Gravels with fines More than 12% fines	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	Sands – 50% or more of coarse Fraction passes No. 4 sieve Coarse, No. 10 to No. 4 Medium, No. 40 to No. 10 Fine, No. 200 to No. 40	Clean Sands Less than 5% fines	SW	WELL GRADED SAND
			SP	POORLY GRADED SAND
		Sands with fines More than 12% fines	SM	SILTY SAND
			SC	CLAYEY SAND
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays – Liquid Limit less than 50 Low to medium plasticity	Inorganic	CL	LEAN CLAY
			ML	SILT
		Organic	OL	ORGANIC CLAY
				ORGANIC SILT
	Silts and Clays – Liquid Limit 50 or more Medium to high plasticity	Inorganic	CH	FAT CLAY
			MH	ELASTIC SILT
		Organic	OH	ORGANIC CLAY
				ORGANIC SILT
Highly Organic Soils	Primarily organic matter, dark in color and organic odor		PT	PEAT

II. DEFINITION OF SOIL COMPONENT PROPORTIONS (ASTM D2487)

			Examples
Adjective Form	GRAVELLY SANDY	>30% to <50% coarse grained component in a fine-grained soil	GRAVELLY LEAN CLAY
	CLAYEY SILTY	>12% to <50% fine grained component in a coarse-grained soil	SILTY SAND
“With”	WITH GRAVEL WITH SAND	>15% to <30% coarse grained component in a fine-grained soil	FAT CLAY WITH GRAVEL
	WITH GRAVEL WITH SAND	>15% to <50% coarse grained component in a coarse-grained soil	POORLY GRADED GRAVEL WITH SAND
	WITH SILT WITH CLAY	>5% to <12% fine grained component in a coarse-grained soil	POORLY GRADED SAND WITH SILT

III. GLOSSARY OF MISCELLANEOUS TERMS

SYMBOLS .... Unified Soil Classification Symbols are shown above as group symbols. A dual symbol “-“ indicates the soil belongs to two groups. A borderline symbol “/” indicates the soil belongs to two possible groups.

FILL.. Man-made deposit containing soil, rock and often foreign matter.

PROBABLE FILL................... Soils which contain no visually detected foreign matter but which are suspect with regard to origin.

DISINTEGRATED ROCK (DR).. Residual materials with a standard penetration resistance (SPT) between 60 blows per foot and refusal. Refusal is defined as a SPT of 100 blows for 2” or less penetration.

PARTIALLY WEATHERED ROCK (PWR)......................... Residual materials with a standard penetration resistance (SPT) between 100 blows per foot and refusal. Refusal is defined as a SPT of 100 blows for 2” or less penetration.

BOULDERS & COBBLES..... Boulders are considered rounded pieces of rock larger than 12 inches, while cobbles range from 3 to 12 inch size.

LENSES................................. 0 to ½ inch seam within a material in a test pit.

LAYERS................................. ½ to 12 inch seam within a material in a test pit.

POCKET .... Discontinuous body within a material in a test pit.

MOISTURE CONDITIONS..... Wet, moist or dry to indicate visual appearance of specimen.

COLOR .... Overall color, with modifiers such as light to dark or variation in coloration.

Schnabel ENGINEERING **TEST BORING LOG**	**Project:** AWI Elm Avenue Storm Drain Relocation Elm Avenue and Veneer Road Portsmouth, Virginia	**Boring Number:**	**B-101**
		Contract Number:	10233031.05
		Sheet:	1 of 2

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: (b H [redacted]

Schnabel Representative: R [redacted] R [redacted]

Equipment: CME-45C (Track)

Method: 2-15/16" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 5/4/12 **Finished:** 5/4/12

X: 12128302.0655 ft **Y:** 3461122.8975 ft

Ground Surface Elevation: 5± (ft) **Total Depth:** 30.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	5/4	8:26 AM	2.0'	---	---
After Drilling ▽	5/4	8:59 AM	1.7'	---	---
Observation Well ▼	5/8	6:19 AM	1.3'	---	---
Observation Well ▼	5/8	12:29 PM	1.3'	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.3	Rootmat and topsoil	FILL	4.5	A		S-1 2+2+1+1 REC=20", 83%	PID = 0 ppm	FILL
1.0	FILL, sampled as silty sand, fine to medium grained sand; moist, brown, estimated 5 - 10% peat, contains roots	FILL	3.8	A			PID = 0 ppm	
2.0	FILL, sampled as clayey sand, fine to medium grained sand; moist, dark brown, contains brick fragments, roots, silty sand pockets	SM	2.8	B1		S-2 2+3+3+4 REC=16", 67%	PID = 0 ppm	ALLUVIUM
4.0	SILTY SAND, fine to medium grained sand; wet, gray		0.8		5	S-3 2+4+2+4 REC=18", 75%	PID = 0 ppm	NORFOLK FORMATION
	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, gray	SP-SM		C1		S-4 4+5+2+1 REC=22", 92%	MC = 25.6% % Passing #200 = 4.2 PID = 0 ppm	
						S-5 1+1+1+1 REC=20", 83%	PID = 0 ppm	
9.0	FAT CLAY WITH SAND; wet, gray, contains organics	CH	-4.2	C2	10		LL = 69 PL = 28 MC = 59.3% % Passing #200 = 74.5 PID = 0 ppm PP <0.25 tsf	
	Change: contains silty sand pockets					S-6 1+1/12"+3 REC=20", 83%	PID = 0 ppm PP <0.25 tsf	
14.5	SILTY SAND, fine to medium grained sand; wet, brown, contains organics	SM	-9.7	C1	15		PID = 0 ppm	
	Change: yellowish brown					S-7 5+4+2+2 REC=22", 92%	PID = 0 ppm	
19.0	SANDY FAT CLAY; wet, light gray, contains organics	CH	-14.2	C2	20		PID = 0 ppm PP =0.25 tsf	
22.0	SILTY SAND, fine to medium grained sand; wet, yellowish brown, estimated 50 - 100% shells	SM	-17.2	C1		S-8 2+1+1+2 REC=1", 4%	PID = 0 ppm PP <0.25 tsf	

(continued)

Schnabel ENGINEERING TEST BORING LOG	Project: AWI Elm Avenue Storm Drain Relocation Elm Avenue and Veneer Road Portsmouth, Virginia	Boring Number: B-101 Contract Number: 10233031.05 Sheet: 2 of 2

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	SILTY SAND, fine to medium grained sand; wet, yellowish brown, estimated 50 - 100% shells *(continued)*	SM		C1				NORFOLK FORMATION *(continued)*
27.0	FAT CLAY WITH SAND; wet, gray, contains organics	CH	-22.2	C2		S-9 1+1+1+1 REC=24", 100%	PID = 0 ppm PP <0.25 tsf	
30.0			-25.2		30			

Bottom of Boring at 30.0 ft.
Boring backfilled with cement/bentonite grout upon completion.
1 1/4" PVC Water Observation Well (W.O.W) installed to 15 ft adjacent to boring upon completion.
W.O.W. readings obtained at low tide (AM readings) and high tide (PM readings).
W.O.W. removed from ground and grout backfilled on 5/8/2012.

Schnabel ENGINEERING TEST BORING LOG	Project: AWI Elm Avenue Storm Drain Relocation Elm Avenue and Veneer Road Portsmouth, Virginia	Boring Number: **B-102** Contract Number: 10233031.05 Sheet: 1 of 2

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: (b [redacted]

Schnabel Representative: [redacted]

Equipment: CME-45C (Track)

Method: 3-7/8" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 5/3/12 **Finished:** 5/3/12

X: 12128497.0197 ft **Y:** 3461070.5663 ft

Ground Surface Elevation: 3± (ft) **Total Depth:** 30.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	5/3	2:43 PM	4.0'	---	---
After Drilling	5/3	3:35 PM	2.8'	---	---
Observation Well	5/8	6:21 AM	1.0'	---	---
Observation Well	5/8	12:32 PM	1.0'	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.2	Rootmat and topsoil		3.0			S-1 3+4+3+3 REC=18", 75%	PID = 0 ppm	FILL
	FILL, sampled as silty sand, fine to medium grained sand; moist, dark brown, contains brick fragments Change: estimated <5% shells, contains wood	FILL		A		S-2 4+3+5+7 REC=22", 92%	PID = 2 ppm	
4.0	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, gray, estimated 5 - 10% shells	SP-SM	-0.8	B1	5	S-3 4+2+4+3 REC=20", 83%	PID = 0 ppm	ALLUVIUM Staining observed from 2 to 10 ft.
6.0	SILTY SAND, fine to medium grained sand; wet, gray, estimated <5% shells	SM	-2.8			S-4 2+1+3+2 REC=18", 75%	PID = 0 ppm	NORFOLK FORMATION
8.0	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, gray, estimated <5% shells	SP-SM	-4.8	C1	10	S-5 2+2+1+1 REC=20", 83%	MC = 22.2% % Passing #200 = 8.3 PID = 1 ppm	Creosote odor detected from 2 to 20 ft.
12.0	FAT CLAY; wet, gray, estimated <5% sand	CH	-8.8	C2		S-6 WOH/24" REC=24", 100%	PID = 0 ppm PP <0.25 tsf	
					15	1, SH REC=24", 100%	LL = 61 PL = 29 MC = 71.6% % Passing #200 = 98.9 PID = 0 ppm PP =0.25 tsf	
						S-7 WOH/24" REC=24", 100%	PID = 0 ppm PP <0.25 tsf	Minor staining observed from 13 to 25 ft.
	Change: contains organics				20	S-8 WOH/24" REC=24", 100%	LL = 76 PL = 31 MC = 67.9% % Passing	

(continued)

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	FAT CLAY; wet, gray, estimated <5% sand *(continued)*						#200 = 97.7 PID = 0 ppm PP <0.25 tsf	NORFOLK FORMATION *(continued)*
	Change: WITH SAND	CH		C2				
						S-9 WOH/18"+1 REC=24", 100%	PID = 0 ppm PP <0.25 tsf	
30.0			-26.8		30			

Bottom of Boring at 30.0 ft.
Boring backfilled with cement/bentonite grout upon completion.
1 1/4" PVC Water Observation Well (W.O.W.) installed to 15 ft adjacent to boring on 5-4-12.
W.O.W. readings obtained at low tide (AM readings) and high tide (PM readings).
W.O.W. removed from ground and grout backfilled on 5/8/2012.

Schnabel Engineering TEST BORING LOG	**Project:** AWI Elm Avenue Storm Drain Relocation Elm Avenue and Veneer Road Portsmouth, Virginia	**Boring Number:** **B-103** **Contract Number:** 10233031.05 **Sheet:** 1 of 2

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: (b [redacted]

Schnabel Representative: (b) (4) [redacted]

Equipment: CME-45C (Track)

Method: 2-15/16" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 5/3/12 **Finished:** 5/3/12

X: 12128566.168 ft **Y:** 3460986.0848 ft

Ground Surface Elevation: 8± (ft) **Total Depth:** 30.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	5/3	12:39 PM	6.0'	---	---
After Drilling	5/3	1:50 PM	See Notes	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.5	Crushed stone; 6-inches crushed concrete		7.0					FILL
	FILL, sampled as silty sand, fine to coarse grained sand; moist, dark brown, contains crushed stone, roots					S-1 5+7+7 REC=16", 89%	PID = 0 ppm	Removed 6-inches of loose crushed concrete by hand.
	Change: brownish green, estimated <5% shells, contains brick fragments, lean clay layers					S-2 3+2+3+3 REC=20", 83%	PID = 0 ppm	
	Change: wood	FILL		A		S-3 7+7+25+29 REC=22", 92%	PID = 38 ppm	Treated wood in split-spoon sampler. Approximately 6 feet of wood encountered. Hard Drilling.
	▽				5	S-4 11+18+20+10 REC=22", 92%	PID = 40 ppm	Creosote odor detected from 4 to 25 ft.
8.0	LEAN CLAY; wet, gray, estimated <5% sand, contains organics	CL	-0.5	B2		S-5 7+6+7+6 REC=16", 67% SS	PID = 11 ppm PP <0.25 tsf PID = 0 ppm	ALLUVIUM
9.0	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, gray, estimated 5 - 10% shells	SP-SM	-1.5	B1	10	S-6 5+4+4+3 REC=6", 25%	PID = 22 ppm	Staining observed from 6 to 15 ft.
13.0	SILTY SAND, fine to medium grained sand; wet, gray	SM	-5.5	C1	15	S-7 2+1+2+2 REC=4", 17%	MC = 30.6% % Passing #200 = 11.8 PID = 64 ppm	NORFOLK FORMATION
17.0	FAT CLAY; wet, gray, estimated <5% sand	CH	-9.5	C2	20	S-8 WOH/24" REC=24", 100%	LL = 76 PL = 32 MC = 77.1% % Passing #200 = 92.1 PID = 1 ppm PP <0.25 tsf	
						S-9 WOH/24" REC=24", 100%	PID = 0 ppm PP <0.25 tsf	Minor staining observed from 23 to 25 ft.

(continued)

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	FAT CLAY; wet, gray, estimated <5% sand *(continued)*	CH		C2		S-10 1/18"+1 REC=24", 100%	PID = 0 ppm PP =0.25 tsf	NORFOLK FORMATION *(continued)*
30.0			-22.5		30			

Bottom of Boring at 30.0 ft.
Boring backfilled with cement/bentonite grout upon completion.
Water observed in casing after completion - 2 ft above ground surface.

Schnabel Engineering — TEST BORING LOG	Project: AWI Elm Avenue Storm Drain Relocation Elm Avenue and Veneer Road Portsmouth, Virginia	Boring Number: **B-104** Contract Number: 10233031.05 Sheet: 1 of 2

Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Contractor Foreman: (b [redacted]

Schnabel Representative: (b) (4) [redacted]

Equipment: CME-45C (Track)

Method: 3-7/8" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)

Dates Started: 5/3/12 **Finished:** 5/3/12

X: 12128669.3499 ft **Y:** 3460861.355 ft

Ground Surface Elevation: 7± (ft) **Total Depth:** 32.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	5/3	8:57 AM	6.0'	---	---
After Drilling ▼	5/3	11:01 AM	6.1'	---	---
Observation Well ▼	5/8	6:15 AM	4.9'	---	---
Observation Well ▼	5/8	12:26 PM	4.9'	---	---

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	FILL, sampled as silty sand, fine to coarse grained sand; moist, grayish brown, contains crushed stone, brick fragments, concrete	FILL		A		S-1. 36+74+47+38 REC=18", 75% SS	PID = 0 ppm PID = 0 ppm	FILL
	Change: brown					S-2 23+34+32+22 REC=20", 83%	PID = 0 ppm	
	Change: wood				5	S-3 13+2+3+4 REC=22", 92%	PID = 11 ppm	Treated wood in split-spoon sampler.
6.0	CLAYEY SAND, fine to medium grained sand; wet, greenish gray, estimated <5% shells	SC	0.6	B1		S-4 2+2+2+2 REC=20", 83%	PID = 3 ppm	ALLUVIUM Creosote odor and staining observed from 4 to 8.5 ft.
8.0	LEAN CLAY WITH SAND; wet, dark grayish green, estimated <5%, contains silty sand pockets	CL	-1.4	B2		S-5 3+4+4+4 REC=24", 100% SS	PID = 8 ppm PP <0.25 tsf PID = 1 ppm PID = 0 ppm	
8.5	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, gray, estimated 5 - 10% shells	SP-SM	-1.9	C1	10 15	S-6 4+3+2+2 REC=18", 75% SS	MC = 21.3% % Passing #200 = 4.5 PID = 3 ppm PID = 0 ppm	NORFOLK FORMATION Creosote odor detected from 4 to 32 ft.
17.0	FAT CLAY; wet, gray, estimated <5% sand	CH	-10.4	C2	20	S-7 1/12"+1+1 REC=24", 100% SS	LL = 74 PL = 29 MC = 70.5% % Passing #200 = 90.9 PID = 1 ppm PP =0.25 tsf PID = 0 ppm	Slight sheen observed from 18 to 30 ft.
						S-8 WOH/12"+1+1 REC=24", 100%	PID = 1 ppm PP =0.25 tsf	

(continued)

Schnabel ENGINEERING TEST BORING LOG	Project: AWI Elm Avenue Storm Drain Relocation Elm Avenue and Veneer Road Portsmouth, Virginia	Boring Number: B-104 Contract Number: 10233031.05 Sheet: 2 of 2

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	FAT CLAY; wet, gray, estimated <5% sand *(continued)*	CH		C2				NORFOLK FORMATION *(continued)*
						S-9 WOH/18"+1 REC=24", 100%	PID = 1 ppm PP <0.25 tsf	
					30	1, SH REC=22", 92%	LL = 66 PL = 31 MC = 66.9% % Passing #200 = 99.2 PID = 3 ppm PP =0.25 tsf	
32.0			-25.4					

Bottom of Boring at 32.0 ft.
Boring backfilled with cement/bentonite grout upon completion.
1 1/4" PVC Water Observation Well (W.O.W.) installed to 15 ft adjacent to boring on 5-4-12.
W.O.W. readings obtained at low tide (AM readings) and high tide (PM readings).
W.O.W. removed from ground and grout backfilled on 5/8/2012.

Schnabel Engineering TEST BORING LOG	Project: Remedial Design AWI Superfund Site Elm Avenue Portsmouth, Virginia	Boring Number: L-2 Contract Number: 08330092 Sheet: 1 of 3

Contractor: Fishburne Drilling, Inc. Chesapeake, Virginia
Contractor Foreman: [redacted]
Schnabel Representative: [redacted]
Equipment: CME-45C (Track)
Method: 2-15/16" O.D. Tri-cone Roller Bit

Hammer Type: Auto Hammer (140 lb)
Dates Started: 9/26/08 **Finished:** 9/26/08

Ground Surface Elevation: 3± (ft) **Total Depth:** 90.0 ft

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered ▽	9/22	11:06 AM	2.0'	---	---
Completion ▼	9/22	3:05 PM	5.3'	---	---
Casing Pulled ▼	9/22	3:09 PM	5.1'	10.0'	76.4'

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
0.2	Rootmat and topsoil		2.8			S-1, SPT 5+8+7+11 REC=24", 100%		FILL
	FILL, sampled as silty sand, fine to coarse grained sand; moist, dark brown, contains crushed stone, contains fat clay layers Change: wet	FILL		A		S-2, SPT 7+7+7+8 REC=24", 100%		
4.0	SILTY SAND, fine to medium grained sand; wet, gray, contains fat clay pockets, estimated <5% shells		-1.0		5	S-3, SPT 6+4+3+3 REC=24", 100%		NORFOLK FORMATION
	Change: contains organics					S-4, SPT 2+1+2+2 REC=24", 100%		
						S-5, SPT 2+2+1+2 REC=24", 100%		
		SM		C1	10			
	Change: estimated 5 - 10% shells					S-6, SPT WOH+1+3+2 REC=24", 100%		
					15			
17.0	FAT CLAY; wet, gray, estimated <5% sand, estimated <5% shells		-14.0			S-7, SPT WOH/12"+1+1 REC=3", 13%	PP <0.25 tsf	
		CH		C2	20			
						S-8, SPT 1+1+1+1 REC=22", 92%	PP <0.25 tsf	

(continued)

Schnabel ENGINEERING **TEST BORING LOG**

Project: Remedial Design AWI Superfund Site
Elm Avenue
Portsmouth, Virginia

Boring Number: L-2
Contract Number: 08330092
Sheet: 2 of 3

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	FAT CLAY; wet, gray, estimated <5% sand, estimated <5% shells *(continued)*	CH		C2				NORFOLK FORMATION *(continued)* lost return water from approximately 25 feet to approximately 70 feet. Used about 300 gallons of water to complete boring.
					30	S-9, SPT 1+1+1+1 REC=24", 100%	PP <0.25 tsf	
					35	S-10, SPT WOH+2+1+1 REC=24", 100%		
					40	S-11, SPT 1+1+1+1 REC=24", 100%	PP <0.25 tsf	
44.0	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, brownish gray	SP-SM	-41.0	C1	45	S-12, SPT 7+6+3+4 REC=24", 100%	PP <0.25 tsf	
	Change: contains organics				50	S-13, SPT 5+5+5+14 REC=24", 100%		
					55	S-14, SPT 3+8+11+12 REC=24", 100%		
						S-15, SPT		

(continued)

Schnabel ENGINEERING TEST BORING LOG	Project: Remedial Design AWI Superfund Site Elm Avenue Portsmouth, Virginia	Boring Number: L-2 Contract Number: 08330092 Sheet: 3 of 3

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOL	ELEV (ft)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	TESTS	REMARKS
	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, brownish gray *(continued)*				60	7+9+9+9 REC=24", 100%		NORFOLK FORMATION *(continued)*
	Change: estimated <5% coarse gravel, contains fat clay lenses	SP-SM			65	S-16, SPT 5+6+6+5 REC=24", 100%		
	Change: reddish gray			C1	70	S-17, SPT 2+2+4+5 REC=24", 100%		
72.0	SILTY SAND, fine to medium grained sand; wet, gray, contains lean clay layers	SM	-69.0		75	S-18, SPT 4+3+5+7 REC=24", 100%		
77.0	POORLY GRADED SAND WITH SILT, fine to medium grained sand; wet, gray Change: dark gray	SP-SM	-74.0		80	S-19, SPT 6+8+9+11 REC=24", 100%		
82.0	SILTY SAND, fine to medium grained sand; wet, greenish gray, estimated 15 - 25% shells	SM	-79.0	D1	85	S-20, SPT 13+13+13+15 REC=24", 100%		YORKTOWN FORMATION
87.0	CLAYEY SAND, fine to medium grained sand; wet, greenish gray, estimated 30 - 45% shells	SC	-84.0		90	S-21, SPT 17+17+19+31 REC=24", 100%		
90.0			-87.0					

Bottom of Boring at 90.0 ft.
Boring backfilled with cement/bentonite grout upon completion.

Appendix H

Ambient Air Standard Calculations

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TABLE 1
AMBIENT AIR MONITORING LEVELS

CAS NO.	Contaminant (a)	RfC (mg/m^3)	IUR (µg/m^3)$^{-1}$	Carcinogenic Screening Level (µg/m^3)	Non-Carcinogenic Screening Level (µg/m^3)	Selected Monitoring Level (µg/m^3) (b)	"Not to Exceed" Offsite Monitoring Level (µg/m^3) (c)
Volatiles							
71-43-2	Benzene	3.0E-02	7.8E-06	2,358	329	329	3,285
108-88-3	Toluene	5.0E+00	N/A	N/A	54,750	54,750	547,500
100-41-4	Ethylbenzene	1.0E+00	2.5E-06	7,358	10,950	7,358	73,584
1330-20-7	Xylenes	1.0E-01	N/A	N/A	1,095	1,095	10,950
Semivolatiles							
83-32-9	Acenapthene	N/A	N/A	N/A	N/A	N/A	N/A
120-12-7	Anthracene	N/A	N/A	N/A	N/A	N/A	N/A
56-66-3	Benz(a)anthracene	N/A	1.1E-04	16.7	N/A	16.7	167
50-32-8	Benzo(a)pyrene	N/A	1.1E-03	1.67	N/A	1.67	16.7
205-99-2	Benzo(b)fluoranthene	N/A	1.1E-04	16.7	N/A	16.7	167
207-08-9	Benzo(k)fluoranthene	N/A	1.1E-04	16.7	N/A	16.7	167
218-01-9	Chrysene	N/A	1.1E-05	167	N/A	167	1672
53-70-3	Dibenz[a,h]anthracene	N/A	1.1E-03	1.67	N/A	1.67	16.7
206-44-0	Fluoranthene	N/A	N/A	N/A	N/A	N/A	N/A
86-73-7	Fluorene	N/A	N/A	N/A	N/A	N/A	N/A
193-39-5	Indeno[1,2,3-cd]pyrene	N/A	1.1E-04	16.7	N/A	16.7	167
90-12-0	Methylnaphthalene, 1-	N/A	N/A	N/A	N/A	N/A	N/A
91-57-6	Methylnaphthalene, 2-	N/A	N/A	N/A	N/A	N/A	N/A
91-20-3	Napthalene	3.0E-03	3.4E-05	541	33	33	329
129-00-0	Pyrene	N/A	N/A	N/A	N/A	N/A	N/A
87-86-5	Pentachlorophenol	N/A	4.6E-06	3,999	N/A	3,999	39,991
Metals							
7440-38-2	Arsenic	1.5E-05	4.3E-03	4.3	0.16	0.16	1.6
7440-36-0	Antimony	N/A	N/A	N/A	N/A	N/A	N/A
7439-89-6	Iron	N/A	N/A	N/A	N/A	N/A	N/A
7439-92-1	Lead (d)	N/A	1.2E-05	1,533	N/A	1,533	15,330
7440-28-0	Thallium	N/A	N/A	N/A	N/A	N/A	N/A

IUR - Inhalation Unit Risk
RfC - Reference Concentration
TEQ - Toxic Equivalent

(a) Contaminants of concern (COCs) are those identified in the Record of Decision (EPA 2007) and lead.

(b) Minimum of the carcinogenic risk or non-carcinogenic effects values. The ambient air concentration a receptor can be exposed continuously throughout the project duration. Conservatively assumes a worker would be exposed during the entire project duration.

(c) Level not to be exceeded off-site for any duration.

(d) Toxicity values are not available for lead. In accordance with the procedures set forth by the USEPA in the Regional Screening Level (RSL) Table, toxicity values for lead are based upon lead acetate and lead subacetate.

Note - Ambient air monitoring levels were calculated using guidance provided in EPA's Risk Assessment Guidance for Superfund (RAGS) and Regional Screening Levels (RSLs) for Superfund Sites (May 2012).

TABLE 2
CALCULATION OF AMBIENT AIR SCREENING LEVELS

Carcinogenic

$$SL_{worker\text{-}air\text{-}c}\ (\mu g/m3) = \frac{TR \times ATr}{EFw \times EDw \times ETwa \times (IUR)}$$

Non-Carcinogenic

$$SL_{worker\text{-}air\text{-}nc}\ (\mu g/m3) = \frac{THQ \times ATr \times CF}{EFw \times EDw \times ETwa \times (1/RfC)}$$

Where:

TR (Target Risk) (unitless) =	1.0E-05	(default)
THQ (Target Hazard Quotient) =	1.0	(default)
ATw (Averaging Time-carcinogen) (days) =	25,550	(=365 days/yr x 70 years; default)
ATw (Averaging Time-non carcinogen) (days) =	152	(= EDr x 365 days/yr))
CF (Conversion Factor) =	1,000	(µg/mg)
EFw (Exposure Frequency) (days/yr) =	100	(assumes 5 days per week for 5 months)
EDw (Exposure Duration) (yr) =	0.42	(site-specific), assumes construction will last 5 months
ETwa (Exposure Time) (hr/hr) =	0.33	(assumes an 8 hour day per 24 hours/day)
IUR (Inhalation Unit Risk) $(\mu g/m^3)^{-1}$=	chemical-specific	
RfC (Reference Concentration) (mg/m^3) =	chemical-specific	
LT (Lifetime) (yrs) =	70	(default)

	Contaminant	VOC	RfC (mg/m^3)	IUR $(\mu g/m^3)^{-1}$	Carcinogenic Screening Level $(\mu g/m^3)$	Non-Carcinogenic Screening Level $(\mu g/m^3)$
Volatiles						
	Benzene	V	3.0E-02	7.8E-06	2.4E+03	3.3E+02
	Toluene	V	5.0E+00	N/A	N/A	5.5E+04
	Ethylbenzene	V	1.0E+00	2.5E-06	7.4E+03	1.1E+04
	Xylenes	V	1.0E-01	N/A	N/A	1.1E+03
Semivolatiles						
	Acenapthene	V	N/A	N/A	N/A	N/A
	Anthracene	V	N/A	N/A	N/A	N/A
	Benz(a)anthracene		N/A	1.1E-04	1.7E+01	N/A
	Benzo(a)pyrene		N/A	1.1E-03	1.7E+00	N/A
	Benzo(b)fluoranthene		N/A	1.1E-04	1.7E+01	N/A
	Benzo(k)fluoranthene		N/A	1.1E-04	1.7E+01	N/A
	Chrysene		N/A	1.1E-05	1.7E+02	N/A
	Dibenz[a,h]anthracene		N/A	1.1E-03	1.7E+00	N/A
	Fluoranthene		N/A	N/A	N/A	N/A
	Methylnaphthalene, 1-	V	N/A	N/A	N/A	N/A
	Methylnaphthalene, 2-	V	N/A	N/A	N/A	N/A
	Napthalene	V	3.0E-03	3.4E-05	5.4E+02	3.3E+01
	Pyrene	V	N/A	N/A	N/A	N/A
	Pentachlorophenol		N/A	4.6E-06	4.0E+03	N/A
Metals						
	Arsenic		1.5E-05	4.3E-03	4.3E+00	1.6E-01
	Antimony		N/A	N/A	N/A	N/A
	Iron		N/A	N/A	N/A	N/A
	Lead (1)		N/A	1.2E-05	1.5E+03	N/A
	Thallium		N/A	N/A	N/A	N/A

N/A = Not available.